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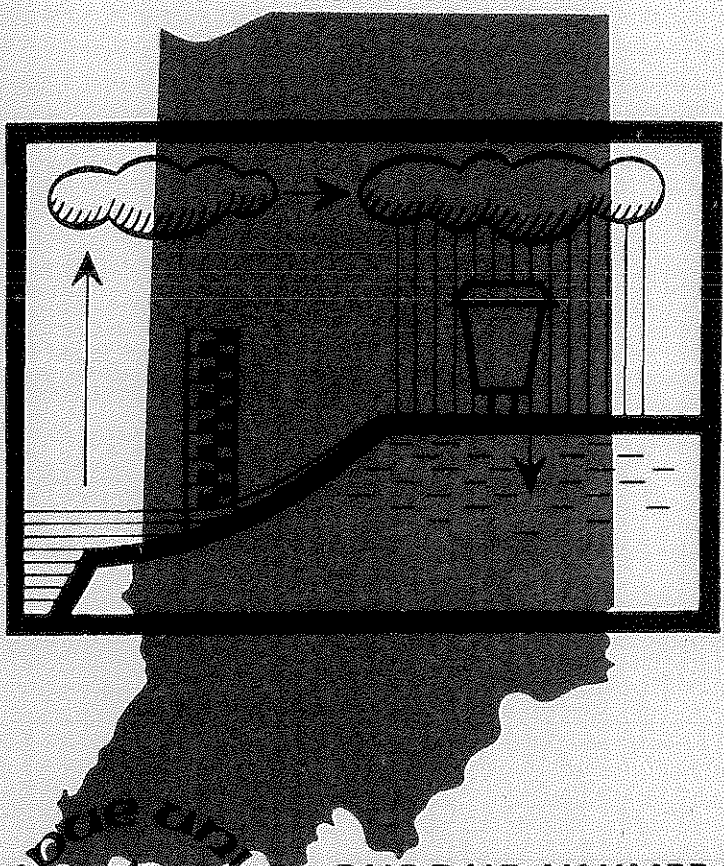

by

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September 1980

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ABSTRACT

The effort which has been expended in the past two decades on the development of urban hydrologic models is quite significant. However, a considerable amount of work still needs to be done on these models in several areas. The present study deals with two of these areas, namely: (1) the optimal estimation of parameters, and (2) the investigation of objective functions in urban rainfall-runoff models.

Optimal parameter estimation methods are needed to eliminate the subjectivity inherent in the trial and error methods. The superiority of the optimal parameter estimation methods over the trial and error methods has not been clearly demonstrated to the practitioners. Such a demonstration would provide the basis for greater confidence in and acceptance of these methods.

In the present study, the regeneration and prediction performances of three urban runoff models, ILLUDAS, SWMM, and MINNOUR, in which optimal parameter estimates are used are compared with the performances of these models in which the parameter values are arbitrarily specified. The results demonstrate that optimal parameter estimates give better regeneration and prediction performance. Another result of the study has been that the complexity of model structure does not automatically guarantee better performance.

Very few investigators have examined the problem of selection of objective functions which are used with the runoff models. Those who have investigated the problem have often used only simple runoff models. Hence, the validity of their conclusions is untested when more complex models such as ILLUDAS are used. ILLUDAS was used in this study to investigate the selection of objective functions.

Two sets of the objective functions were tested by using the data from the Upper Ross-Ade and Oakdale Avenue watersheds. The sum of the squared deviations between the observed and calculated hydrograph ordinates has been the most frequently used objective function in the past and the results of the present study show that this objective function gives the best overall performance.

CHAPTER I

INTRODUCTION

There has been considerable research and development activity in urban stormwater management during the past two decades. This is mainly due to recognition of the facts that the methodologies used in designing drainage systems have been cursory, have not improved over time and that large expenditures are involved in design, planning, and construction of the drainage systems. The total national expenditure involved in urban drainage systems is conservatively estimated to be two billion dollars. The storm sewer design followed by most practicing engineers is based on the rational formula (ASCE (1976)). Need for caution in the use of rational method has been emphasized many investigators (Schaake et al. (1967), Yen et al. (1974)). However, because of its simplicity and because it gives conservative designs, the rational method is still being used in drainage design.

The research needs in urban hydrology and drainage design have been explicitly discussed only during the last 15 years or so (McPherson and Mangan (1975)). During this time, several federal and state agencies have sponsored programs in urban water resources research. A number of urban rainfall-runoff models have been developed and many of these have been released for public use.

The more important of these models are ILLUDAS (Terstriep and Stall (1974)), SWMM (Metcalf and Eddy et al. (1971)), STORM (Corps of Engineers (1977)), and the model developed by the United States Geological Survey (Alley et al. (1980)). There are many discussions of these models (McPherson (1975)). The effort which has been expended on the development of these models is quite significant. However, a considerable amount of work still needs to be done on these models in several areas. The present study deals with two of these areas, namely: (1) the optimal estimation of parameters, (2) the investigation of objective functions in urban rainfall-runoff models.

Runoff models are being used to serve two major purposes. Firstly, they are used to design new drainage systems. Secondly, they are used to evaluate existing systems. In designing a new drainage system, appropriate design parameters are needed so that the runoff at various points may be computed and a proper design may be arrived at. Evaluation of an existing drainage system may include an analysis of the system deficiencies and, if needed, generation of alternatives to improve it.

Outputs computed by using urban runoff models will be erroneous and hence would provide poor results if invalid or inappropriate input parameter values are used in these models. If measured rainfall-runoff and other necessary data are available for the drainage system which is being evaluated, then that data may be used to calibrate the model. Only a calibrated model can be used effectively to evaluate and improve the system.

So far, urban rainfall-runoff models are calibrated mainly by trial and error methods. In these trial and error methods, the model parameters are systematically varied until the model output compares "favorably" with the observed data according to a preselected criterion. The trial and error methods are guided by parametric sensitivity analyses which are used to identify those parameters estimated by trial and error.

The trial and error calibration methods have the obvious disadvantages of being subjective and expensive (Overton (1977)). Even if the computational burdens and costs of trial and error methods are acceptable, the resulting set of parameter estimates may not be the best. Consequently, automatic parameter estimation methods are needed to eliminate the subjectivity inherent in trial and error methods and to reduce computational costs. Although several rainfall-runoff models have parameter estimation methods built into them, optimal parameter estimation methods have not been commonly used in urban drainage design models. Furthermore, the superiority of the optimal parameter estimation methods over the trial and error methods has not been clearly demonstrated to the practitioners. Such a demonstration would provide the basis for greater confidence in and acceptance of these models.

Finally, the design of urban drainage systems and their costs are dependent on the parameter estimates of the models. It is important to use appropriate parameter values in the urban runoff models so that drainage system costs may be accurately estimated.

In view of the foregoing discussion, the first major objective of the study is to develop optimal parameter estimation methods for two of the more important urban runoff models. The regeneration and prediction performance of the urban runoff models in which optimal parameter estimates are used are compared with the performance of the models in which the parameters are simply selected, perhaps based on "engineering judgement." This portion of the study is discussed in Chapter III.

The optimal parameter estimation in urban runoff models needs an index of agreement between the observed data and that computed by using the model. This index is the objective function. The selection of an objective function depends on the emphasis which the modeler wishes to place on particular characteristics of the model. For example, the objective function must emphasize the total runoff volume under the design hydrograph if the primary purpose of the model is to aid in the design of detention storage facilities. On the other hand, the objective function must have a measure of the peak and shape of the design hydrograph if it is to be used in designing drainage systems. For general investigation of drainage systems, a combined form which consists of two components, the shape and the volume, may be suitable. However, the selection of the objective function for the parameter estimates of a runoff model is a subjective process as various choices of the objective function are possible. The optimal parameter estimates are obviously optimal only with respect to of the objective function used to estimate the parameters.

Procedures of selecting an objective function for hydrologic models have been investigated, although not in great depth. Dawdy and O'Donnell (1965) defined an objective function as the sum of the squared deviations between the observed and calculated runoffs for their catchment model and suggested other criteria be used in future studies. One of the most recent and complete investigations concerning selection of objective functions was performed by Diskin and Simon (1977). The rainfall-runoff models used by Diskin and Simon (1977) were simple. Hence, the validity of their conclusions is untested when more complex models such as ILLUDAS, are used. Consequently, the second major objective of this study is to investigate the selection of objective functions to be used with urban runoff models. ILLUDAS was used to establish guidelines for selecting objective functions for urban drainage system design. This objective function analysis is discussed in Chapter IV. A general discussion of results and a set of conclusions are presented in Chapter V.

CHAPTER II

DATA USED IN THE STUDY

2.1 Introduction

Data from two watersheds, the Upper Ross-Ade Watershed and the Oakdale Avenue Basin. were used in this study. Data from the Upper Ross-Ade Watershed were used in optimal estimation of parameters of urban runoff models and the objective function analysis. Data from the Oakdale Avenue Basin were used in the investigation of optimal parameter estimation methods and in the investigation of objective functions. Oakdale Avenue data were also used to test the MINNOUR model. A description of these watersheds and the data from these watersheds follows.

2.2 Upper Ross-Ade Watershed

The Upper Ross-Ade Watershed is located in West Lafayette, Indiana. Its location is shown in Fig. 2.1 and some of the details of the watershed are given in Fig. 2.2. The Upper Ross-Ade Watershed has an area of 29 acres. It is mainly residential and is relatively uniform in character. The watershed extends in a generally north-south direction

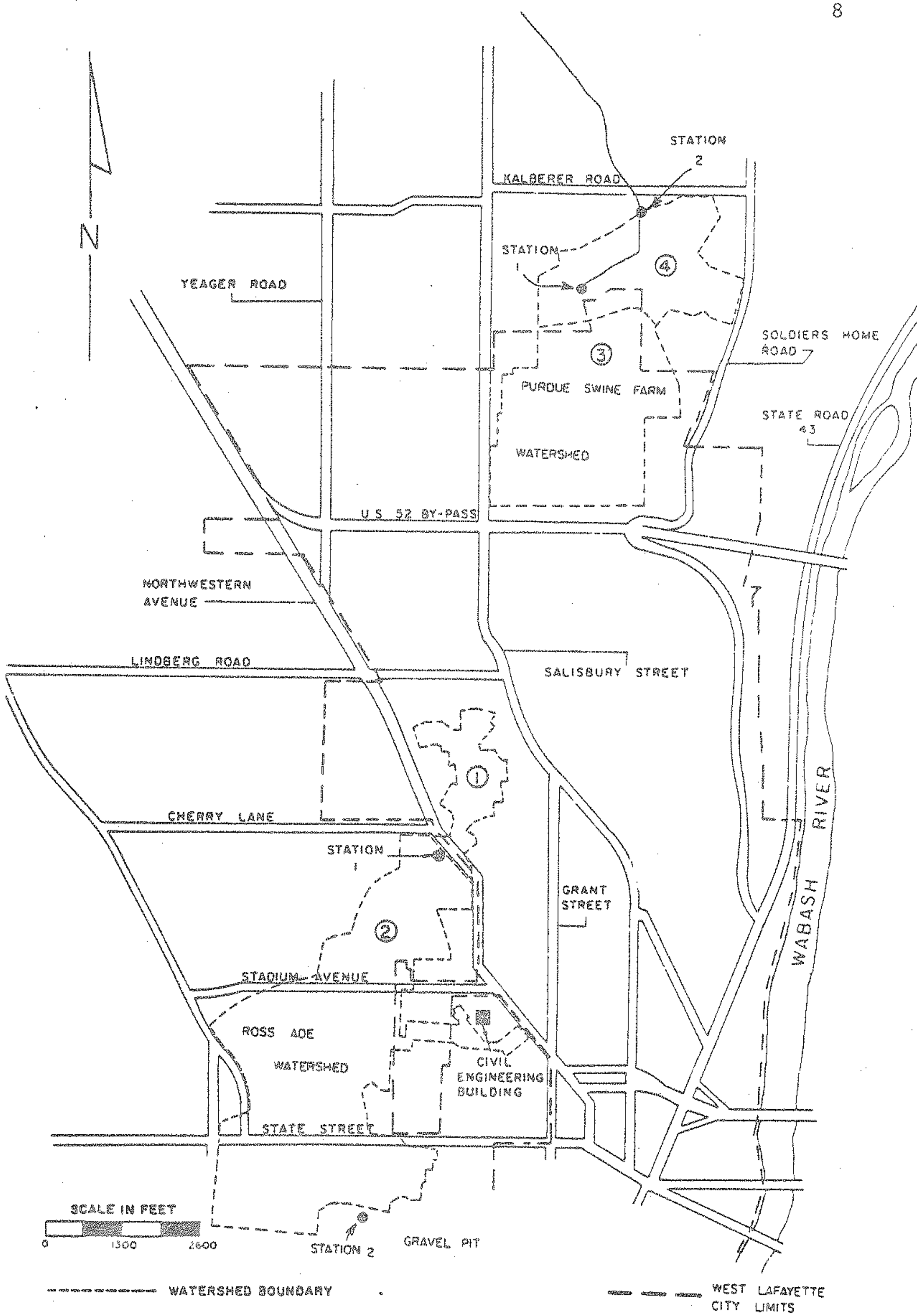


Figure 2.1 Location of the Upper Ross-Ade Watershed (From Sarma (1970))

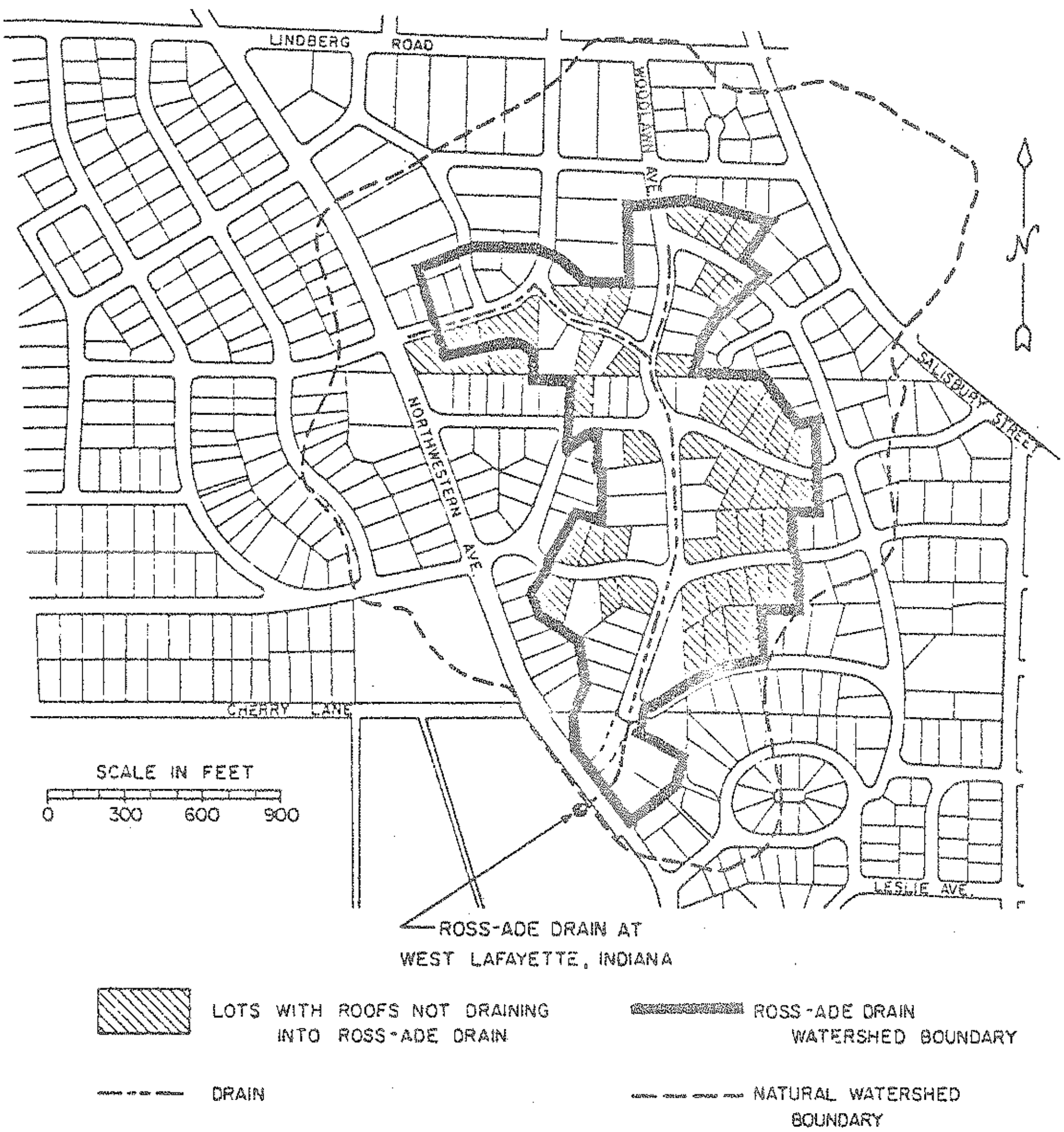


Figure 2.2 Upper Ross-Ade Watershed (From Han and Delleur (1979))

and about 38 percent of its area is impervious to infiltration (Vician and Delleur (1966)). The soil types in the watershed ranges from Crosby silt loam in the lower levels to Miami silt loam on the steeper portions to Eel silt loam on the uplands as shown in Fig. 2.3 and summarized in Table 2.1 (Hossain et al. (1974)). Details of the gaging station, instrumentation, data acquisition, and physiographic characteristics of the watershed have been described by Sarma (1970).

For evaluating an existing system, the physical layout as well as various details of the system are needed. These are usually obtained from a set of detailed drainage system plan and profile drawings. Such details for the Upper Ross-Ade watershed were obtained from the Office of the City Engineer, West Lafayette, Indiana. A schematic representation of the drainage system is shown in Fig. 2.4. The length, slope, and diameter of each pipe segment are listed in Table 2.2.

2.3 Oakdale Avenue Basin

Reliable rainfall-runoff data are available from Oakdale Avenue Basin in Chicago. The Oakdale Avenue Basin is located in a residential section about six miles northwest of downtown Chicago as shown in Fig. 2.5. A plan view of the basin is shown in Fig. 2.6. The Oakdale Avenue basin is smaller than the Upper Ross-Ade Watershed. It is

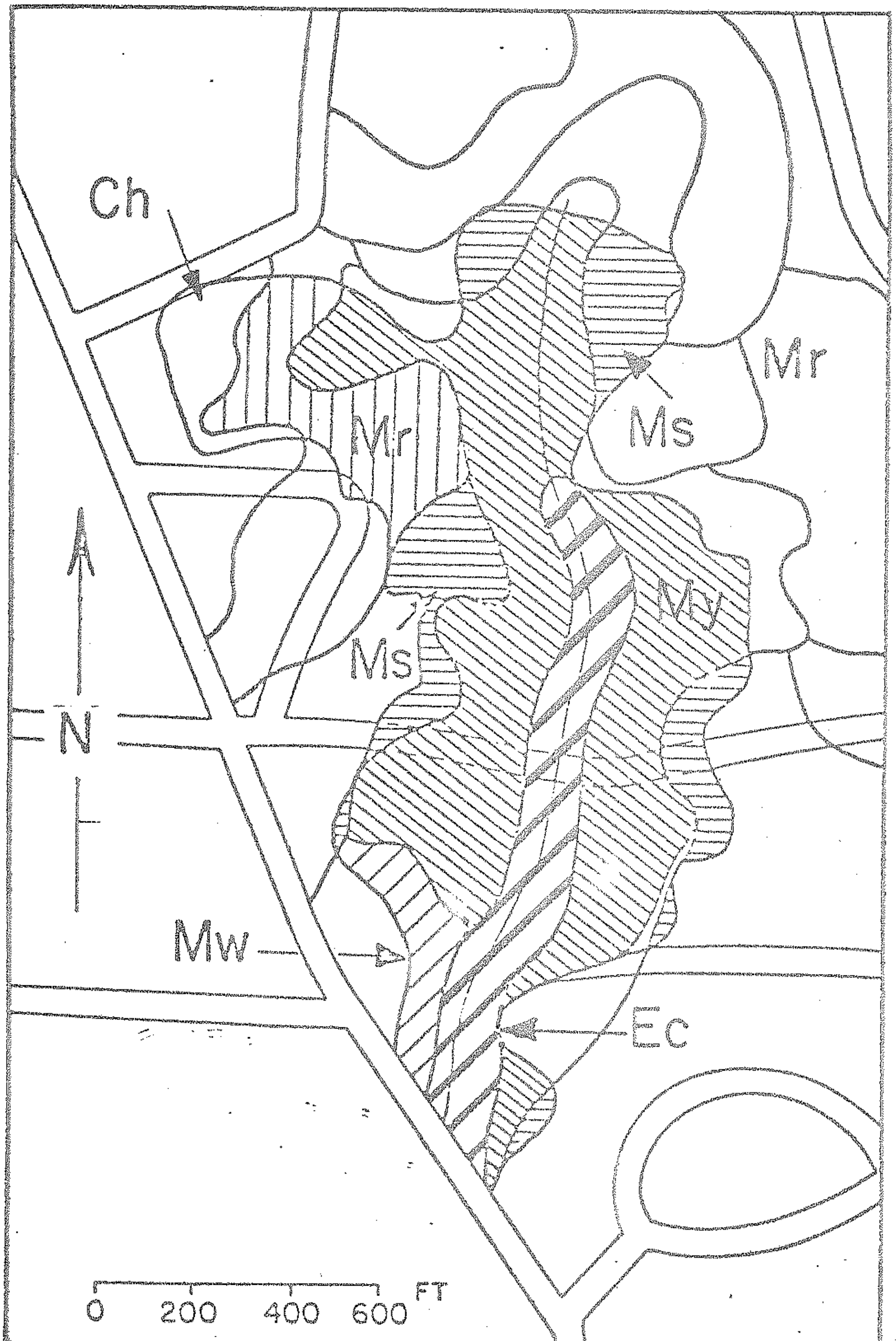


Figure 2.3 Soil Map—Upper Ross-Ade Watershed (From Hossain et al. (1974))

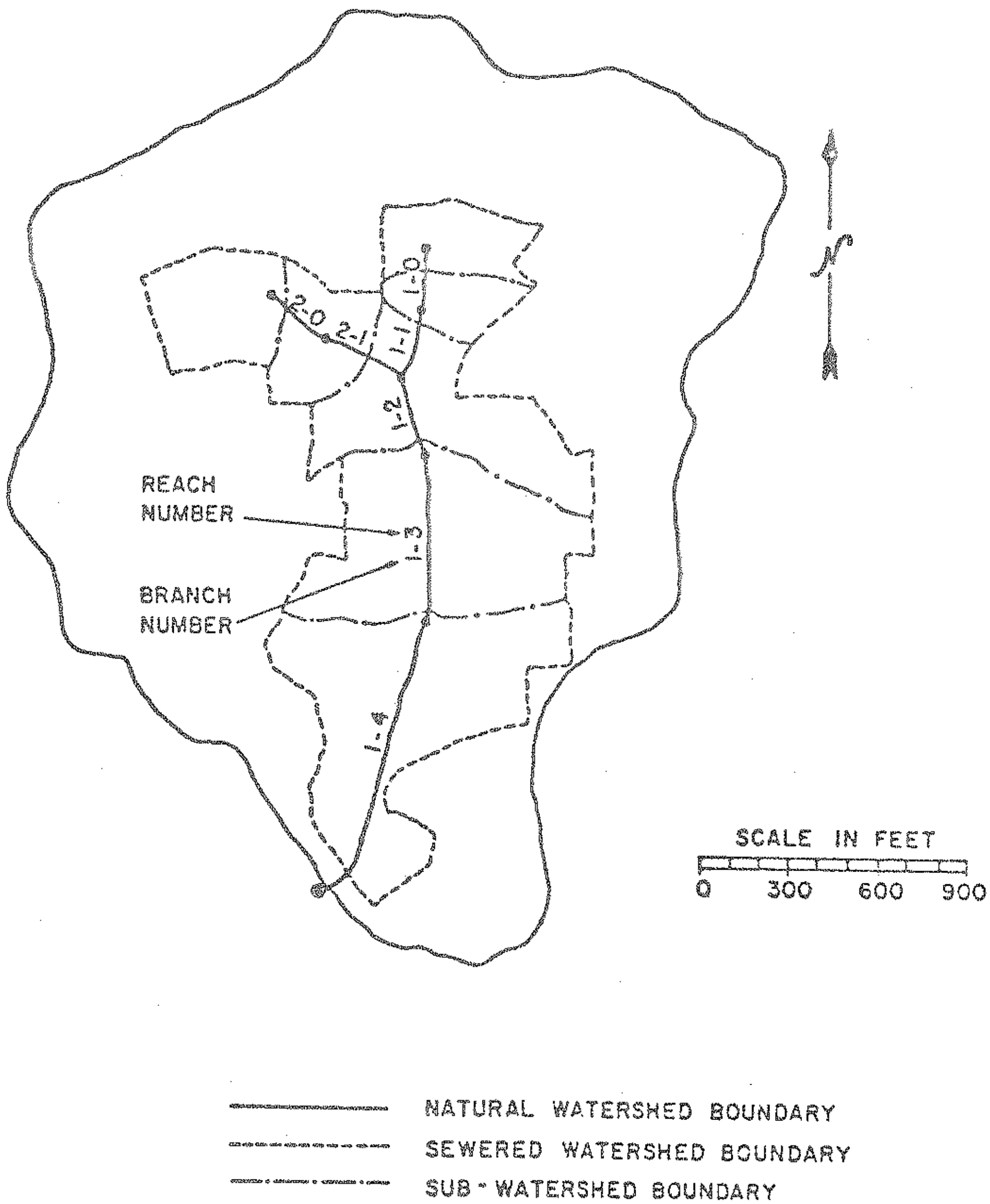


Figure 2.4 Schematic Representation of the Drainage System in Upper Ross-Ade Watershed

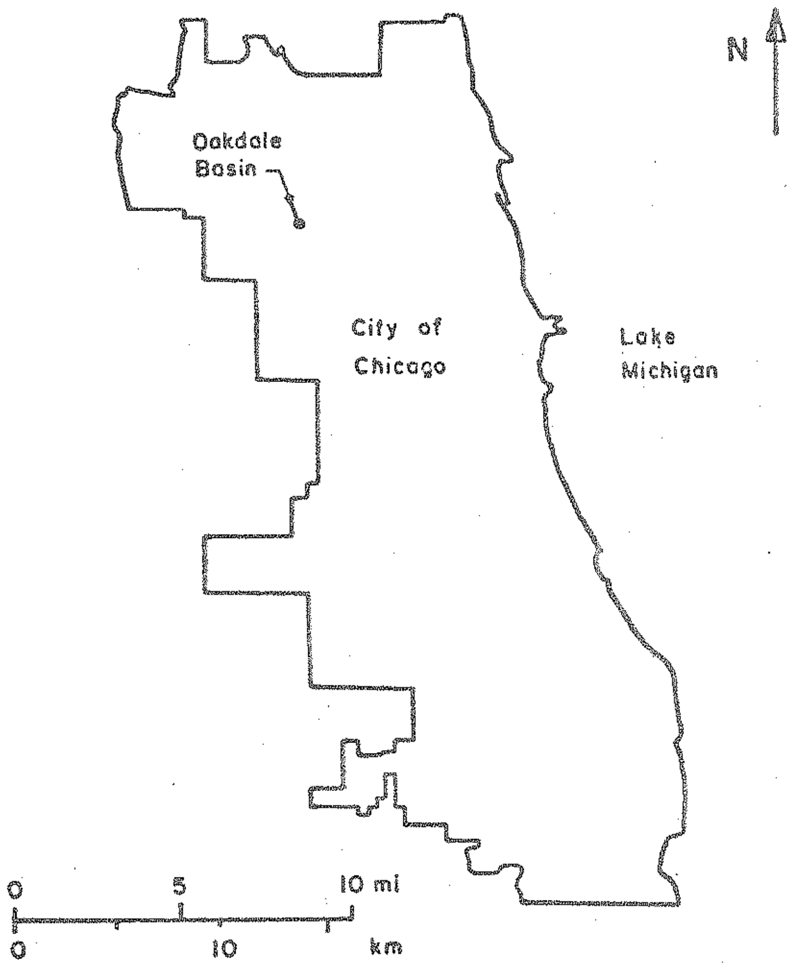


Figure 2.5 Location of the Oakdale Avenue Basin (From Chow and Yen (1976))

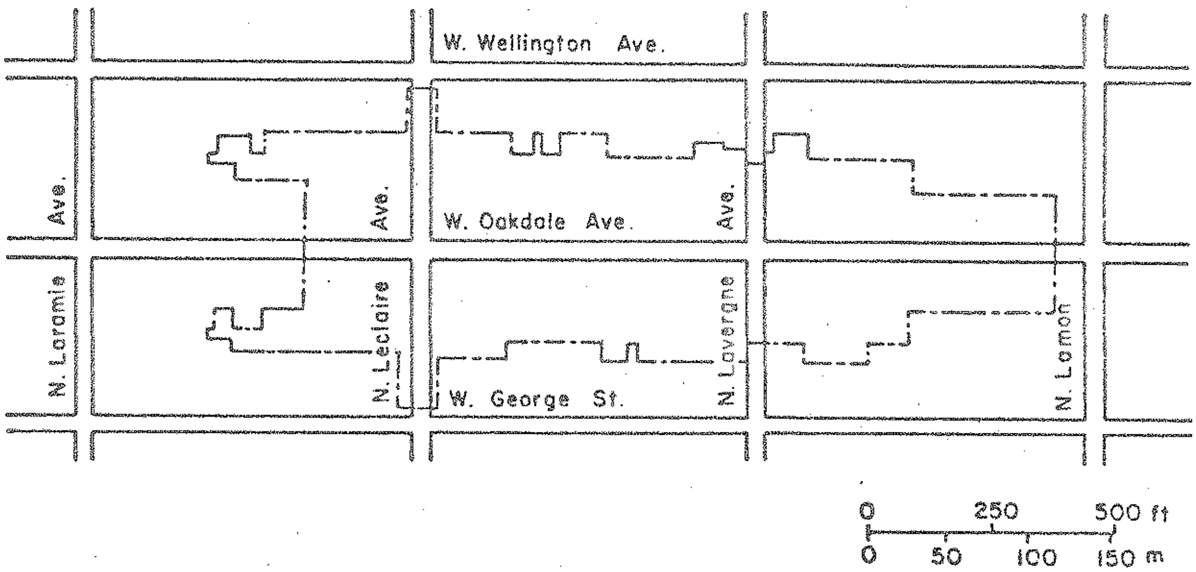


Figure 2.6 Plan View of the Oakdale Avenue Basin (From Chow and Yen (1976))

Table 2.1 Soil Types in Upper RossAde Watershed (From Hossain, et al. (1974))

Symbol	Soil Type	Depth of Top Layer (in)	Area (%)
Ec	Eel silt loam 03% slope	6.0 to 8.0	16
Ms	Miami silt loam 38% slope	3.0 to 8.0	13
Mr	Miami silt loam 38% slope	7.0	12
Ch	Crosby silt loam 03% slope	7.0	5
Mw	Miami silt loam 1225% slope	3.0 to 8.0	5
Mv	Miami silt loam 1225% slope	8.0 to 12.0	49

Table 2.2 Physical Characteristics of Pipe Segments in the Upper Ross-Ade Watershed (From Han and Delleur (1979))

Branch	Reach	Length (ft)	Slope (%)	Diameter (in)
1	0	240	3.4	18
1	1	260	3.4	21
2	0	250	4.8	15
2	1	280	4.0	18
1	2	240	2.0	30
1	3	536	2.8	36
1	4	860	1.5	36

approximately 2½ blocks long and one block wide and has a area of 12.9 acres. The Oakdale Avenue Basin consists of 54.6% pervious and 45.4% impervious areas. Eighty-Eight percent of the impervious area drains directly to combined sewers and twelve percent of the drainage area drains indirectly. A detailed breakdown of drainage area according to Tucker (1968) is given in Table 2.3.

The existing 30-inch diameter, reinforced concrete, combined sewer in the basin was installed in 1958. It drains east along W. Oakdale Avenue for little more than two blocks into a 10.5′×10.5′ concrete combined trunk sewer. The combined trunk sewer drains north into the North Branch of the Chicago River.

Rainfall-runoff data from the Oakdale Avenue Basin have been periodically measured and recorded from 1959 to 1964 by the Bureau of Engineering, Chicago Department of Public Works. A tipping bucket raingage was used to measure rainfall. Runoff data were measured with a Simplex 30-inch type "S" parabolic flume. The storm events for which accurate data were collected are listed in Tucker (1968). Additional information regarding the Oakdale Avenue Basin can be found elsewhere (Chow and Yen (1976), Papadakis and Preul (1972), Terstriep and Stall (1969)).

Characteristics of some of the typical storms of the Upper Ross-Ade Watershed and the Oakdale Avenue Basin are

Table 2.3 Breakdown of Pervious and Impervious Surfaces in Drainage Area (From Tucker (1968))

Impervious Area:	
Draining Directly to Combined Sewer:	
Houses	2.52 arces
Streets	1.58 acres
Alleys	0.58 acres
Garages	0.27 acres
Sidewalks:	
To Street	0.11 acres
To Alley	0.09 acres
SUBTOTAL =	5.15 acres
Draining Indirectly to Combined Sewer:	
Public Walks	0.57 acres
Private Walks	0.15 acres
SUBTOTAL =	0.72 acres
Pervious Areas (grassed, etc.):	
SUBTOTAL =	7.05 acres
Total Drainage Area=	12.92 acres

Table 2.4 Storm Characteristics of Upper Ross-Ade Watershed and Oakdale Avenue Basin

Watershed	Date of Storm (M/D/Y)	Rainfall Duration (min)	Total Rainfall (in)	Rainfall Intensity (in/hr)	Runoff Duration (min)	Total Runoff (in)	Peak Runoff (cfs)	Runoff Coefficient (-)
Upper Ross-Ade Watershed	7/18/70	345	2.19	0.38	624	0.33	15.9	0.15
	7/30/70	427	2.21	0.31	468	0.41	8.4	0.19
	9/04/70	205	1.34	0.39	492	0.23	12.3	0.17
	6/24/70	14	0.45	1.93	144	0.06	10.3	0.13
Oakdale Avenue Basin	5/19/59	115	0.68	0.35	190	0.26	6.5	0.38
	7/02/60	70	0.36	0.31	95	0.14	4.6	0.39
	7/26/60	295	0.79	0.16	425	0.42	4.3	0.53
	9/18/60	200	0.71	0.21	260	0.39	5.1	0.55
	10/14/60	135	0.25	0.11	190	0.12	4.5	0.48
	4/17/63	265	0.74	0.17	360	0.35	6.5	0.47
	4/19/63	90	0.50	0.33	155	0.22	11.6	0.45
	4/29/63	360	1.35	0.23	360	0.75	6.7	0.56
	6/19/63	160	0.30	0.11	245	0.24	7.8	0.80
	8/02/63	100	0.65	0.39	180	0.25	5.9	0.38
	9/22/64	40	0.42	0.63	120	0.18	4.4	0.43
	7/29/59	60	0.53	0.53	95	0.32	9.2	0.60
	8/24/63	130	0.35	0.16	155	0.16	2.9	0.46

listed in Table 2.4. Part of the data is used for the regeneration study and the remainder is used for the prediction study. The data listed here are from typical storms and are considered accurate.

2.4 Parameter Values Used in the Study

Some of the parameters of the hydrologic models were held constant when the models are applied to different watersheds. These parameters were held constant either because variation in them did not drastically affect the output, or they could be measured accurately, or they could be easily obtained from readily available references. These parameters are listed in Table 2.5. The parameter values used in the Complex optimization scheme in MINNOUR are listed in Table 2.6. The parameters which were optimally estimated are discussed in the following chapters.

Table 2.5 Constant Input Used in the Study

	ILLUDAS	
Input Parameter	Upper Ross-Ade Watershed	Oakdale Avenue Basin
Area (acres)	29.1	12.9
Soil Group	B	B
Manning's n	0.013	0.013
Time Increment (min)	5	5
Routing Scheme	Time Lag	Time Lag
	SWMM	
Time Step (min)	5	5
Erosion to be Modeled	No	No
Rainfall Interval (min)	5	5
Manning's n	0.013	0.013
No. of Subcatchments	7	14
Quality modeled	No	No
	MINNOUR	
No. of Subcatchments		1
No. of Raingage Station		1
Base Flow (cfs)		0.00
Hydraulic Length of Longest Course (ft)		1660
Watershed Slope (%)		21.2
Impervious area (%)		45
Zero Depression Storage Impervious Area to Total Impervious Area (%)		68
Time Step (min)		5
No. of Time Step		100
Infiltration Process Used		1
Hydrograph Method		0
Volume of Soil Moisture Capacity (%)		31.0
Volume of Gravitational Water (%)		11.4
Factor of Hydraulic Length Improved Future Condition		1
Factor of Impervious Area Future Condition		1
Final Infiltration Rate (in/hr)		0.50

Table 2.6 Parameter Values Used in the Complex Optimization Scheme

Parameter Notation	Description	Value
N	Number of explicit independent input parameters	5
M	Number of sets of constraints	5
K	Number of points in the complex	10
ITMAX	Maximum number of iterations	300
IC	Number of implicit parameters	0
ALPHA	Reflection coefficient	2.00
BETA	Convergence parameter	0.100
GAMMA	Convergence Parameter	5
CCOEFF	Contraction coefficient	0.50
BETH	Expansion coefficient	2.50
SHRINK	Shrinkage coefficient	0.50
STRTSK	Convegence parameter	10.0
DELTA	Explicit constraint violation correction	0.0001
MAXEX	Number of max. expansion operation	10
MAXCN	Number of max. contraction operation	1
MAXSK	Number of max. shrinkage operation	0

CHAPTER III

OPTIMAL PARAMETER ESTIMATION IN URBAN RUNOFF MODELS

3.1 Introduction and Statement of the Problem

Urban drainage design models are being used both to design new drainage systems and to evaluate existing systems. The design of urban drainage systems involves locating and sizing storm sewers, flow control and diversion structures, and treatment facilities to meet specified performance criteria. In designing drainage systems, appropriate design storm rainfall, soil cover characteristics, and the drainage system configuration are specified. Urban runoff models are used to compute the time distribution of runoff and peak flows at various points in the drainage system so that components of the system may be sized.

Evaluation of an existing drainage system may include an analysis of existing system deficiencies and examination of alternatives to improve the system. However, results from evaluation studies will be misleading if invalid or inappropriate input data are used in the runoff model. If measured rainfall, runoff and other necessary data are available for the drainage system which is being evaluated, then these data may be used to calibrate urban runoff model. The calibrated runoff model may then be used to simulate system performance for other events with an eye toward improving the system. At present, trial and error methods are used to calibrate urban runoff models. In these trial and error procedures, individual model parameters are systematically varied one at a time, until the model output compares "favorably" with the measured data according to a preselected criterion. The trial and error calibration may be aided by sensitivity analysis of model parameters in the sense that only the more sensitive parameters may be varied.

The trial and error calibration of urban runoff models is subjective and expensive (Overton (1977)). Consequently automatic parameter estimation methods are needed to eliminate the subjectivity and also to reduce computational cost. Although many runoff models have parameter estimation procedures built into them, optimal parameter estimation methods have not yet been commonly used with urban runoff models. One of the objectives of the present study is to develop optimal parameter estimation procedures for some of the more important urban runoff models.

Although the subjectivity inherent in the trial and error methods are eliminated when optimal parameter estimation methods are used, demonstration of the superiority of the optimal parameter estimation methods over the trial and error methods would be beneficial. Such a demonstration would provide the basis for greater confidence in these methods. Consequently, in the present study, the results obtained by using the optimal parameter estimates are compared with those in which the parameters are selected according to other criteria such as those suggested by the user's manuals of the various models.

Finally, there is another important reason to develop optimal methods of parameter estimation of urban runoff models. This relates to economic studies of urban drainage systems. Urban runoff models are frequently used in economic analysis of urban drainage systems (Miller (1978), Grigg and O'Hearn (1976), Froise and Burges (1978)). However, the size of drainage systems and hence their costs are dependent on the input data and on model parameter estimates. Consequently, it is important to use appropriate parameter values in the runoff models, at least for the more important parameters, the results from which may then be used to estimate drainage system costs. The best method of estimating these parameters, especially when rainfall-runoff data from the system are available, is to calibrate the model by using the observed data and optimal parameter estimation methods.

In view of the above discussions, the first objective of this chapter is to develop optimal parameter estimation methods for some of the more important urban runoff models.

The second objective is to compare the performance of the urban runoff models in which optimal parameter estimates are used with the performance of models in which "judgmental" parameter values are used. Both regeneration and prediction performances of the models are tested. Finally, the third objective is to compare the prediction and regeneration performances of some of the urban runoff models with optimal parameter estimates.

The optimization technique used in the study is the modified Rosenbrock's optimization method (Rosenbrock (1968)). The parameters of the urban runoff models, the Illinois Urban Drainage Area Simulator (ILLUDAS) developed at the Illinois State Water Survey (Terstriep and Stall (1974)) and the Runoff Block of the Storm Water Management Model (SWMM) (Metcalf and Eddy et al. (1971)) were estimated by Rosenbrock's method. The parameter estimation method was tested by using data from the Upper Ross-Ade Watershed in West Lafayette, Indiana (Sarma (1970), Hossain et al. (1974)) and from the Oakdale Avenue Basin in Chicago, Illinois (Tucker (1968)).

The third model investigated in the present study is the Minnesota Optimized Urban Runoff Model (MINNOUR) (Chu (1978)). This is a mathematical model with a built-in automatic optimization method. The optimization method used in MINNOUR is the modified Box-Complex scheme (Box (1965), Chu (1978)).

3.2 Literature Review

Two types of parameters may be distinguished in runoff models in general, and in urban drainage design models in particular. The first of these comprises the parameters which may be obtained from watershed topographical maps or aerial photographs. These include the areas, slopes, and imperviousness, etc. As accurate values of these parameters may be readily obtained they need not be estimated and consequently they are not considered further in this study.

Values of the second type of parameters are either assumed or they are obtained if rainfall-runoff data are available. Trial and error methods have been used to estimate the parameters of urban drainage design models despite the obvious disadvantages of these methods. This may be due to the fact that various optimization techniques have been applied to estimate the parameters of the hydrologic models only since the late sixties. One of the earliest attempts to estimate parameters optimally in a rainfall-runoff model was by Dawdy and O'Donnell (1965). They investigated the sensitivity of their rainfall-runoff model and used the modified Rosenbrock's method to compute the optimal parameter estimates of their model. Liou (1970) developed a computer program to semi-automatically estimate the parameters in the modified Stanford Watershed Model. Chapman (1970) successfully used the Simplex method of Nelder and Mead (1965) to estimate the parameters of a rainfall-runoff model. Wood and Sutherland (1970) estimated the parameters in the Stanford Model IV by the steepest descent method. Ibbitt and O'Donnell (1971) examined nine leading optimization techniques to estimate the parameters of the conceptual model developed by Dawdy and O'Donnell (1965) and concluded that Rosenbrock's method was the most effective parameter estimation method. A modified version of Rosenbrock's method was used by Carrigan (1973) to calibrate the model developed at U. S. Geological Survey. Johnston and Pilgrim estimated the parameters of the Boughton model (Boughton (1965,1966)) by the Simplex method (Nelder and Mead (1965)) and by the steepest descent method (Fletcher and Powell (1963)) and preferred the Simplex method. Kite (1978) developed a hydrologic model in which the Rosenbrock's method was used. Wildermuth and Yeh (1979) applied a least squares parameter estimation algorithm to the Los Angeles County Flood Control District Runoff Forcast Model (Wildermuth (1976)). The most recent study in optimal parameter estimation in a rainfall-runoff model is that by Sorooshian (1978) and by Sorooshian and Dracup (1980). They developed a maximum likelihood procedure to estimate model parameters. Sorooshian (1980) also compared two direct search algorithms used in calibration of rainfall-runoff models and concluded that for estimation of sensitive parameters in rainfall-runoff models, Rosenbrock's method is to be preferred.

The improvements in regeneration or prediction performance of rainfall-runoff models brought about by the optimal parameter estimation have not been widely reported. This may explain, to a certain extent, the lack of acceptance of the optimal parameter estimation methods by design engineers. Also, although optimal parameter estimation methods have been recently used in rainfall-runoff models, the optimization routines are not used in two of the commonly used urban runoff models, namely, ILLUDAS and the Runoff Block of SWMM. So far, the parameters of these models are estimated by using improvised methods, most frequently by trial and error methods. Consequently, methods of optimal parameter estimation were developed for these models and the improvements in the model performance are demonstrated in the present study.

3.3 ILLUDAS

The ILLUDAS model was originally developed for simulation of urban runoff from single storm events. The runoff from both grassed and paved areas are estimated in these models.

The paved area hydrograph is calculated in ILLUDAS by the linear time-area method. The travel time is found by Manning's equation and the initial abstraction is subtracted from the initial hyetograph increment to obtain the paved area supply rate (PASR). The final hydrograph is determined by assuming that the first increment of runoff results from the first increment of area and first PASR. The second runoff area increment results from the first PASR on the second area and the second PASR on the first area increment and so on. These calculations may be expressed in the matrix notation as follows.

$$[Q]=[PASR][PA] \tag{3.1}$$

in which

$$[Q]=\begin{bmatrix} Q_1 \\ Q_2 \\ \cdot \\ \cdot \\ Q_n \end{bmatrix} \qquad [PA]=\begin{bmatrix} PA_1 \\ PA_2 \\ \cdot \\ \cdot \\ PA_n \end{bmatrix}$$

$$[PASR]=\begin{bmatrix} PASR_1 & 0 & 0 & 0 \\ PASR_2 & PASR_1 & 0 & 0 \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ PASR_n & \cdot & \cdot & PASR_1 \end{bmatrix}$$

The grassed area hydrograph which consists of runoff from the grassed area and the supplemental paved area is considered by ILLUDAS but it was ignored in the RRL method (Watkins (1962)) from which ILLUDAS evoloved. The procedure is quite similar to that of the paved area hydrograph. The travel time is found by Izzard's equation (Izzard (1946)) and linearity is assumed for the time-area curve. The grassed area supply rate (GASR) is determined by adding the hyetograph to the supplemental paved area runoff (SPARO). These areas typically include roofs, sidewalks, etc. The infiltration in the watershed is calculated by using Horton's equation (Horton (1940), Chow (1964)). The grassed area hydrographs are then derived by:

$$[Q]=[GASR][GA] \tag{3.2}$$

in which

$$[Q]=\begin{bmatrix} Q_1 \\ Q_2 \\ \cdot \\ \cdot \\ Q_n \end{bmatrix} \qquad [GA]=\begin{bmatrix} GA_1 \\ GA_2 \\ \cdot \\ \cdot \\ GA_n \end{bmatrix}$$

$$[GASR]=\begin{bmatrix} GASR_1 & 0 & 0 & 0 \\ GASR_2 & 0 & 0 & 0 \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ GASR_n & \cdot & \cdot & GASR_1 \end{bmatrix}$$

Once the grassed and paved area hydrographs are determined, they are added together and to the upstream hydrographs. The sewer is sized at this stage by using Manning's equation and the peak flows. The smallest allowable commercially available pipe which can carry the flow equal to or larger than the peak flow is selected. The hydrograph is routed further downstream to another design point by one of two routing methods. They are, (1) the time shift or time lag method, and (2) storage routing in which the implicit solution of the continuity equation is used. The process is repeated until the final outlet is reached.

In the design mode, ILLUDAS is used to calculate the required channel or sewer diameters. In using ILLUDAS it is necessary to subdivide the watershed into subbasins served by different sewer branches and reaches. The data needed to use ILLUDAS include the rainfall magnitude and temporal distribution, the antecent moisture condition, the hydrologic soil group, and the percentage of paved and grassed areas. In the evaluation mode, the pipe, culvert or open ditch dimensions, their lengths, slopes, values of Manning's n, etc. are also needed. Maps, aerial photos and drainage system drawings of the basin may be needed to extract the branch, reach, and subbasin information. Even so, the data requirements of ILLUDAS are moderate compared to those of other urban runoff models.

The user's manual and the computer deck of ILLUDAS may be obtained from the Illinois State Water Survey (Terstriep and Stall (1974)). The program has been tested and the results have been found to be satisfactory. Burke (1979) compared ILLUDAS with two other methods on two watersheds and concluded that ILLUDAS is accurate, flexible, easy to apply, and moderate in data requirements.

ILLUDAS is being continuously improved. In December, 1978, the flow routing algorithm was expanded by the Illinois State Water Survey. The latest version of ILLUDAS dated October, 1979, has two routing options, a time shift of the entire hydrograph, and storage routing using the implicit solution method. The water quality algorithm of SWMM is also being adapted in a version of ILLUDAS known as QUAL-ILLUDAS (Terstriep et al. (1978)). Han and Delleur (1979) modified ILLUDAS for continuous simulation and added a single storm quality simulation routine to the ILLUDAS model.

3.4 SWMM

The Storm Water Management Model of the U. S. Environmental Protection Agency is a comprehensive model for simulation of urban storm runoff. It is capable of simulating runoff quality and quantity as well as dry weather flow, treatment facilities, associated costs, and receiving water quality. The model consists of a main control and service block, an executive block, and four computational blocks: (1) Runoff Block, (2) Transport Block, (3) Storage Block, and (4) Receiving Water Block. The

Runoff Block simulates both runoff quantity and quality by accepting a rainfall hyetograph and by accounting for rainfall, infiltration, detention, overland flow, and gutter flow. The flow is routed through the sewer system in the Transport Block. The quantity and quality constituents are routed through the sewers by using a finite difference form of the modified Manning's equation. The Storage Block permits inclusion of storage elements and treatment facilities in the model scheme. Costs associated with the construction and operation of these facilities are computed. The Receiving Block simulates the quantity and quality effects of stormwater on receiving water bodies. The SWMM model is being widely used and good results have been reported by many investigators (Heeps and Mein (1974), Jewell et al. (1974), Marsalek et al. (1975)). The SWMM is also being improved.

In the present study, some of the parameters in ILLUDAS and the Runoff Block of SWMM are optimally estimated. The parameters in other blocks of the SWMM model may also be estimated by using methods such as those discussed below. However, it is well known that the water quality data needed to optimally estimate the water quality parameters are not easily available. The computational complexity also increases with the number of parameters. Consequently only the parameters in the Runoff Block of SWMM are optimally estimated in the study.

3.5 MINNOUR

The MINNOUR model (Chu (1978)) is a simple mathematical urban runoff model with a built-in optimal parameter estimation routine. An important feature of this model is its simplicity.

The Thiessen polygon method is used in MINNOUR to calculate the average rainfall if there is more than one raingage in the watershed. The constants related to pervious and impervious area abstractions are input to the model. One of the three infiltration models, the modified Holtan's infiltration equation, shifted Horton's infiltration equation, and the Horton's infiltration equation, may be selected to estimate the infiltration. The modified Holtan's infiltration equation is derived from Holtan's equation which is given as Eq. (3.3) (Holtan (1961))

$$f=a(S-F)^{n}+f_{c} \tag{3.3}$$

where f: rate of infiltration in inches/hour

a: a constant depends on the density of vegetation and stage of growth; a=1 in MINNOUR model

S: storage in overlaying horizons computed as all of the porosity above the wilting point, in inches

F: cumulative infiltration in inches of water already stored in excess of the wilting point in the soil at time t, in inches

n: an exponent, n=1.4 in MINNOUR model

f_c: final constant rate of infiltration in inches/hour

The modified Holtan's equation (Huggins and Monke (1966, 1970)) is based on original Holtan's equation and the following assumptions: (1) the drainage rate is zero when the moisture content is less than the field capacity, (2) the soil is assumed to be completely saturated when the infiltration rate becomes constant, (3) the drainage rate is equal to the infiltration rate when infiltration rate becomes a constant, (4) when the water content is between field capacity and saturation, the drainage rate is computed according to Eqs. (3.4) and (3.5):

$$f=f_c+a\left(\frac{S-F}{T_p}\right)^n \tag{3.4}$$

$$\frac{f_s}{f_c}=1-\left(\frac{PIV}{GWC}\right)^3 \tag{3.5}$$

in which T_p: total porosity of the soil above the impeding layer, in inches

GWC: maximum volume of gravitational water, in inches

PIV: unsaturated volume, in inches

f_s : drainage rate, in inches/hour

f, f_c, a, S, F, and n are defined eariler.

The Horton's infiltration equation (Horton (1940), Chow (1964)) is:

$$f=f_c+(f_o-f_c)e^{-kt} \qquad (3.6)$$

where f : rate of infiltration capacity, in inches/hour

f_c : final infiltration rate, in inches/hour

f_0: initial infiltration rate, in inches/hour

k : constant

t : time, in hours

The advantage of Horton's equation is its simplicity. However, the parameters are dependent on soil type and soil moisture content and it is very difficult to estimate their values.

The shifted Horton's infiltration equation is based on a time-offset method. At the beginning of the rainfall, if rainfall intensity is less than the infiltration capacity, then the infiltration curve is offset by time t_0. Two conditions must be satisfied at the offset time. These are, (1) rainfall intesity is equal to infiltration rate and (2) accumulated precipitation volume must be equal to accumulated infiltration volume.

After the infiltration rate is calculated, the rainfall excess is obtained by subtracting detention storage and infiltration capacity from the rainfall. The runoff hydrograph is then generated by SCS triangular unit hydrograph method (SCS (1972)).

The time to peak T_p and peak flow Q_p are computed according to Eqs. (3.7) and (3.8).

$$T_p=(\Delta D/2)+L \qquad (3.7)$$

$$Q_p=(484A)/T_p \qquad (3.8)$$

where ΔD: the duration of unit excess rainfall, in hours

L: the lag time, in hours

A: the drainage areas, in square miles

The lag time L can be computed by either one of the following two methods.

(1) SCS modified curve number (CN) method

The lag time is given by eq. (3.9) and is mainly applicable to agricultural watersheds of less than 2000 acres. However, it can be modified and used for urban areas by using suitable curve numbers.

$$L = \frac{l^{0.8}\,(s+1)^{0.7}}{1900Y^{0.5}} \tag{3.9}$$

where L: lag time, in hours

l: hydraulic length of watershed, in feet

$$s = \frac{1000}{CN} - 10 \tag{3.10}$$

where CN: curve number

Y: average watershed land slope, in percent

(2) SCS hydrograph method

In the SCS hydrograph method, the empirical formula for lag time given in eq. (3.11) which is applicable to average natural watershed conditions is used.

$$L=0.6T_c \qquad (3.11)$$

in which L: lag time, in hours and

T_c : time of concentration, defined as the time for runoff to travel from the furthermost point in the watershed to the design point, in hours

The other option in MINNOUR to obtain T_p and Q_p is based on Synder's method in which the 30 minute duration unit hydrograph parameters are estimated by Espey's empirical equation (Espey et al. (1969)).

$$T_q=222\times L_0{}^{0.38}S^{-0.45} \qquad (3.12)$$

$$Q_{pq}=23580A^{0.99}T_q{}^{-1.0}, \text{ if } I<20\% \qquad (3.13)$$

$$T_q=143L_0{}^{0.43}S^{-0.06}I^{-0.32} \qquad (3.14)$$

$$Q_{pq}=40490A^{0.96}T_q{}^{-1.1}, \text{ if } I\geqq 20\% \qquad (3.15)$$

where L_0: longest water course, in miles

S : channel slope, defined as

$$S = \frac{(\text{elev. at } 0.9L) - (\text{elev. at } 0.1L)}{0.8L}$$

A : drainage area, in square miles

I : percent of impervious drainage area

The regression equations (3.12) to (3.15) were derived by Espey et al. (1969) by analyzing the data from 37 basins. The 30-minute-duration unit hydrograph is transformed to another duration T_{pq} to compute T_p and Q_p.

$$T_{pq}=T_q-\frac{\text{Duration}}{2} = T_q - \frac{0.5}{2} \qquad (3.16)$$

$$T_p=\frac{T_{pq}-0.25\times\text{Duration}}{0.955} = \frac{T_{pq}-0.25\times0.5}{0.955} \qquad (3.17)$$

$$Q_p=\frac{Q_{pq}\times T_{pq}}{T_p} \qquad (3.18)$$

Once T_p and Q_p are determined, the ordinates of the unit hydrograph are derived by proportional scaling from T_p and Q_p values. The total base time, however, is determined from eq. (3.19) which is based on the assumption of a triangular unit hydrograph.

$$T_B = \frac{2\times2323200\times A}{60\times Q_p} \tag{3.19}$$

in which T_B : total base time, in minutes

A : area, in square miles

Q_p: peak discharge, in cfs

The ordinates of the triangular unit hydrograph can now be estimated. The ordinates of the storm hydrograph are calculated by using effective rainfall data.

3.6 Selection And Optimal Estimation of Parameters in ILLUDAS and SWMM

Those parameters which can be accurately estimated by using readily available data and parameters which are not very sensitive to the model output need not be estimated by numerical methods. Consequently, the first step in optimal estimation of parameters in urban runoff models is to select the parameters in these models which must be optimally estimated. This selection is generally based on the results of parametric sensitivity analysis of models.

The second step in optimal parameter estimation is the selection of objective or criterion functions. Different objective functions may give different parameter estimates, hence proper choice of objective functions is important.

The objective functions selected must reflect the importance given by the modeler to different aspects of the use of the model.

The third step is to choose a good optimization method to estimate the parameters and to properly design the computational sequences. If the parameter estimation method is not appropriately selected, the resulting parameter estimates may not be reliable or robust. If the computational sequences are not properly designed the computational expenditure may become unreasonable. The above three steps of optimal parameter estimation are discussed in this section.

3.6.1 Selection of Parameters

Two criteria were used in the present study to select parameters to be optimally estimated. The criteria are: (1) the parameter should be difficult to measure, and (2) the model output should be sensitive to variations of that parameter. The parmeters which may be ralatively easily estimated are separated out first. Of the remaining parameters, only those whose variation affects the model output significantly are selected for optimal estimation. The results of sensitivity analyses of SWMM and ILLUDAS were used to select these parameters.

Han and Delleur (1979), in their sensitivity analysis of ILLUDAS, concluded that the antecedent moisture condition and the soil group were sensitive parameters of that model. Based on the results of this study, six parameters of ILLUDAS were selected for optimal estimation and these are listed in Table 3.1.

The parametric sensitivity of SWMM model has been analyzed by several investigators (Graham et al. (1974), Huber et al. (1975), and Jewell (1974)). Results from these studies have been tabulated by Jewell et al. (1978). The parametric sensitivity of the SWMM model was analyzed as a part of the present study by using the Upper Ross-Ade Watershed data. Typical results of this sensitivity analysis study are given in Table 3.2. The available results of parametric sensitivity analysis of SWMM are summarized in Table 3.3.

Although the impervious area and width of subcatchment, which is really the physical width of overland flow, are sensitive parameters in SWMM, they are measurable and hence are not included in the set of estimated parameters. The infiltration parameters, the maximum and minimum infiltration rates, and the decay rate of the infiltration curve are not easily measurable and they significantly affect the output. Consequently, these parmeters were optimally estimated. The depression storages in pervious and impervious areas were also optimally estimated. The parameters in SWMM which are optimally estimated in the present study are listed in Table 3.4.

Table 3.1 Parameters Which Were Optimally Estimated in ILLUDAS

ABSTRT	An initial abstraction in inches from rain fall on the paved portion of the basin to account for surface wetting and depression storage
DEPG	An initial abstraction in inches from rain fall on the grassed portion of the basin to account for depression storage
FI	Infiltration accumulated in soil mantle in inches at start of rainfall
FO	Initial infiltration rate in inches per hour
FC	Final constant infiltration rate in inches per hour
KIF	Infiltration decay constant in hour^{-1}

Table 3.2 Typical Results from Parametric Sensitivity Analysis of SWMM. (Upper Ross-Ade Watershed Data)

Parameter	Range	7/19/1974	8/23/1974
		Change in Volume	Change in Volume
Impervious Area Manning's n	1:100	1:1.005	1:0.994
Pervious Area Manning's n	1:150	1:1.00	1:1.00
Impervious Area Depression Storage	1:200	1:0.804	1:0.405
Pervious Area Depression Storage	1:50	1:1.00	1:1.00
Maximum Rate of Infiltration	1:4	1:1.00	1:1.00
Mininum Rate of Infiltration	1:150	1:1.00	1:1.00
Infiltration decay constant	1:6	1:1.00	1:1.00

Table 3.3 Summary of Results of Sensitivity Analysis of SWMM Model

	Graham et al. (1974)		Huber et al. (1975)		Jewell (1974)		Present Study	
Parameter	Range	Total Volume	Range	Total Volume	Range	Total Volume	Range	Total Volume
Impervious Area as a Percentage	7:1	2.01:1	–	–	4:1	3.58:1	–	–
Pervious Area Minimum Infiltration Rate	1:10	1.82:1	1:150	1.75:1	–	–	–	–
Characteristic Width	1:79	2.00:1	–	–	2:1	1.03:1	–	–
Pervious Area Manning's n	1:4	1.36:1	1:150	1.00:1	–	–	1:150	1:1.00
Impervious Area Depression Storage	1:4	1.24:1	1:200	1.22:1	1:7	1.28:1	1:200	1.86:1
Pervious Area Depression Storage	1:4	1.02:1	1:50	1:1	–	–	1:50	1:1
Impervious Area Manning's n	1:3	1.02:1	1:100	1.02:1	1:2.3	1.02:1	1:100	1:1

There are five parameters which were optimally estimated in the MINNOUR model. (1) SDEPTH, the soil moisture control depth which is defined as impeding strata where the actual infiltration rate equals the final infiltration rate, in inches; (2) SMI, initial soil moisture content, in %; (3) DSPERV, depression storage for the pervious area, in inches; (4) DSIMPV, depression storage for the impervious area, in inches; and (5) TC, time of concentration of the watershed, in minutes.

Table 3.4 Parameters Which Were Optimally Estimated in SWMM

WW(7)	Detention storage of the impervious area in inches
WW(8)	Detention storage of the pervious area in inches
WW(9)	Maximum infiltration rate in inches/hour
WW(10)	Minimum infiltration rate in inches/hour
WW(11)	Infiltration decay constant in $second^{-1}$

3.6.2 Objective Function

Objective functions are indices of agreement between observed and estimated runoff. The choice of objective functions is important as it influences the parameter estimates (Dawdy and Thompson (1967)). The estimated parameters are optimal only with respect to the objective function used. The choice of objective functions in rainfall-runoff models and their effect on the parameters and model outputs have been investigated by various investigators (Clark (1973), Johnston and Pilgrim (1976), Diskin and Simon (1977)).

The objective function used in the present study is of the form given in Eq. (3.20).

$$U=\omega\sum_{i=1}^{n}(O_i-C_i)^2+(1-\omega)(VO-VC)^2 \qquad (3.20)$$

In Eq. (3.20) O_i and C_i are respectively the ith observed and calculated hydrograph ordinates, VO and VC are the observed and calculated runoff volumes, n is the number of ordinates, and ω is a weighting factor which varies between 0 and 1. The importance attached to discharge rate or runoff volume may be varied by an appropriate choice of ω. For example, if the storm sewer size is more important than accurate runoff volume estimation, ω may be selected to be closer to unity. In the preseny study, ω was set equal to 0.5. The ω value of 0.5 may not indicate that equal importance is attached to both runoff rate and volume when the number of hydrograph ordinate n is small. However, when n is large, the importance attached to both components become equal. In the present study, the hydrographs have long enough durations or n was quite large.

3.6.3 Optimization

ILLUDAS and the Runoff Block of SWMM were rearranged as subroutines so that the parameters could be optimally estimated with maximum effeciency. The physical data and the parameters which were not optimally estimated were held constant. Rosenbrock's hill-climbing technique (Rosenbrock (1960)) was used to estimate the parameters of ILLUDAS and the Runoff Block of SWMM. The parameters were optimally estimated in two steps. In the first step, the values of each of the parameters were systematically varied until no further improvement resulted in the objective function. This procedure is repeated for all the parameters. In the second step, the Rosenbrock's hill-climbing method is used with the parameters estimated by the first step as the initial estimates to obtain the final optimal parameter estimates.

The hill-climbing method starts with initial parameter estimates α_i and step lengths β_i, i=1, 2, $\cdots$, N, where N is the number of parameters to be estimated. The objective function is evaluated by using initial estimates and the hydrologic model. After each evaluation of the objective function, the objective function value is checked to see whether it has improved, the parameter limits are checked to see if they are violated, and the constraints are checked to see if they are satisfied. If the objective function value has improved (success), then the step length β_i is multiplied by a factor δ, $\delta \geqq 1.0$. If the objective function value has decreased (failure) then β_i is multiplied by a factor δ, $0 \leqq \delta \leqq 1.0$.

The same procedure is followed for all variables until a "success" and a "failure" are encountered in succession. The axes are then rotated to define a new set of N orthogonal search directions by the Gram-Schmidt orthogonalization procedure. This procedure is terminated when the convergence criterion is met. If constraints are violated, the trial is considered a failure. If the estimate is in the boundary zones then the objective function is modified. The boundary conditions are defined as follows:

Lower Zone: $G_k \leqq \alpha_k \leqq (G_k + (H_k - G_k) \cdot \gamma)$ (3.21)

Upper Zone: $H_k \leqq \alpha_k \leqq (H_k - (H_k - G_k) \cdot \gamma)$ (3.22)

$k = 1, 2, \cdots, M$

where M are the number of constraints. In the present study N=M, H_k and G_k are the upper and lower bounds respectively. γ is recommended to be equal to 10^{-4} (Kuester and Mize (1973)).

The objective function is modified by using Eq. (3.23) where F^+ is the current objective function value at a point where the constraints are satisfied and at which the boundary zones are not violated.

$$F_n=F_0-(F_0-F^+)(3\lambda-4\lambda^2+2\lambda^3) \tag{3.23}$$

F_0 is the old objective function value, λ is defined by Eqs. (3.24) and (3.25), and the computation is continued as before.

$$\text{Lower Zone:} \quad \lambda=\frac{G_k+(H_k-G_k)\cdot\gamma-\alpha_k}{(H_k-G_k)\cdot\gamma} \tag{3.24}$$

$$\text{Upper Zone:} \quad \lambda=\frac{\alpha_k-(H_k-(H_k-G_k)\cdot\gamma)}{(H_k-G_k)\cdot\gamma} \tag{3.25}$$

The flow charts showing the optimal parameter estimation scheme are given in Figs. 3.1 and 3.2. The optimization procedure used in the study is the same as that used by Kite (1970).

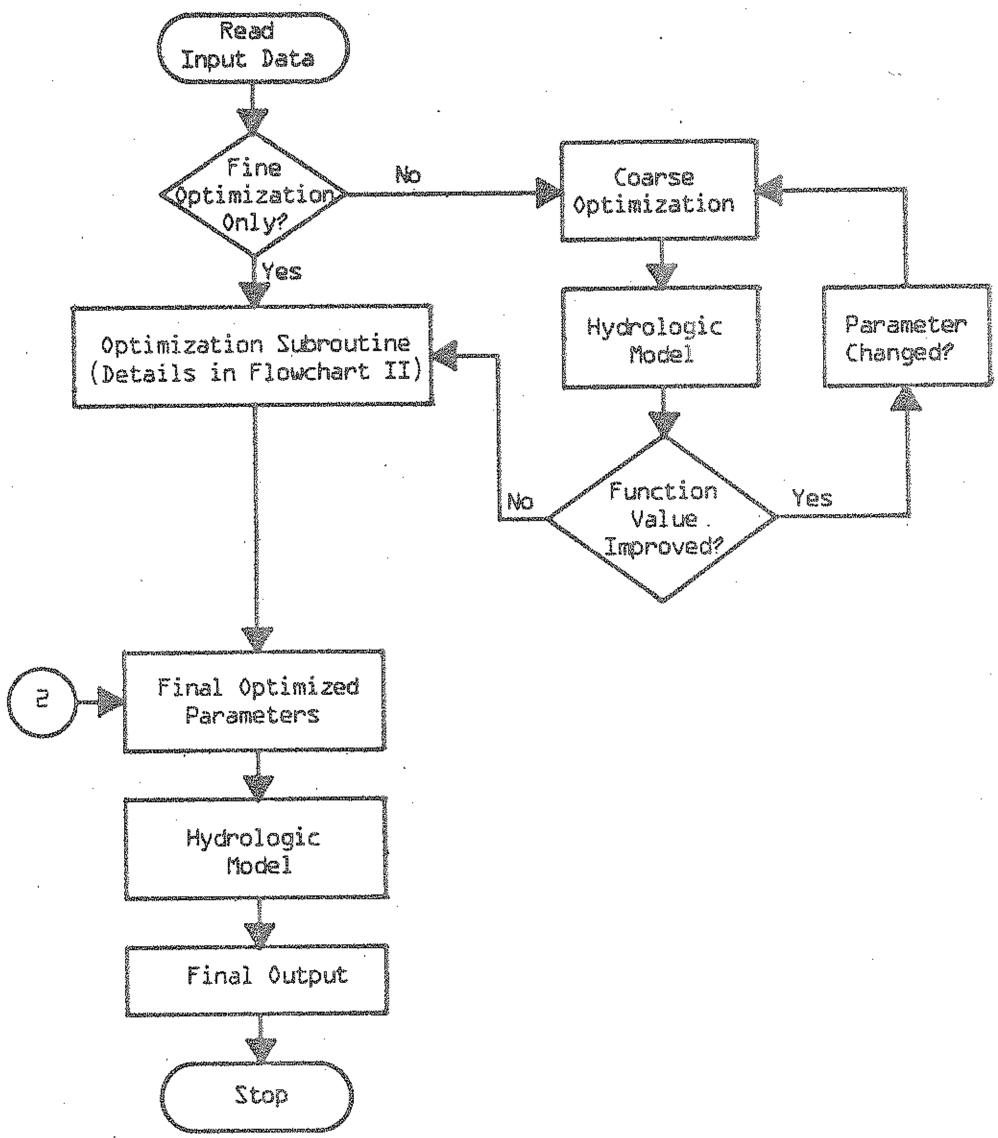


Figure 3.1 Optimization Scheme Used

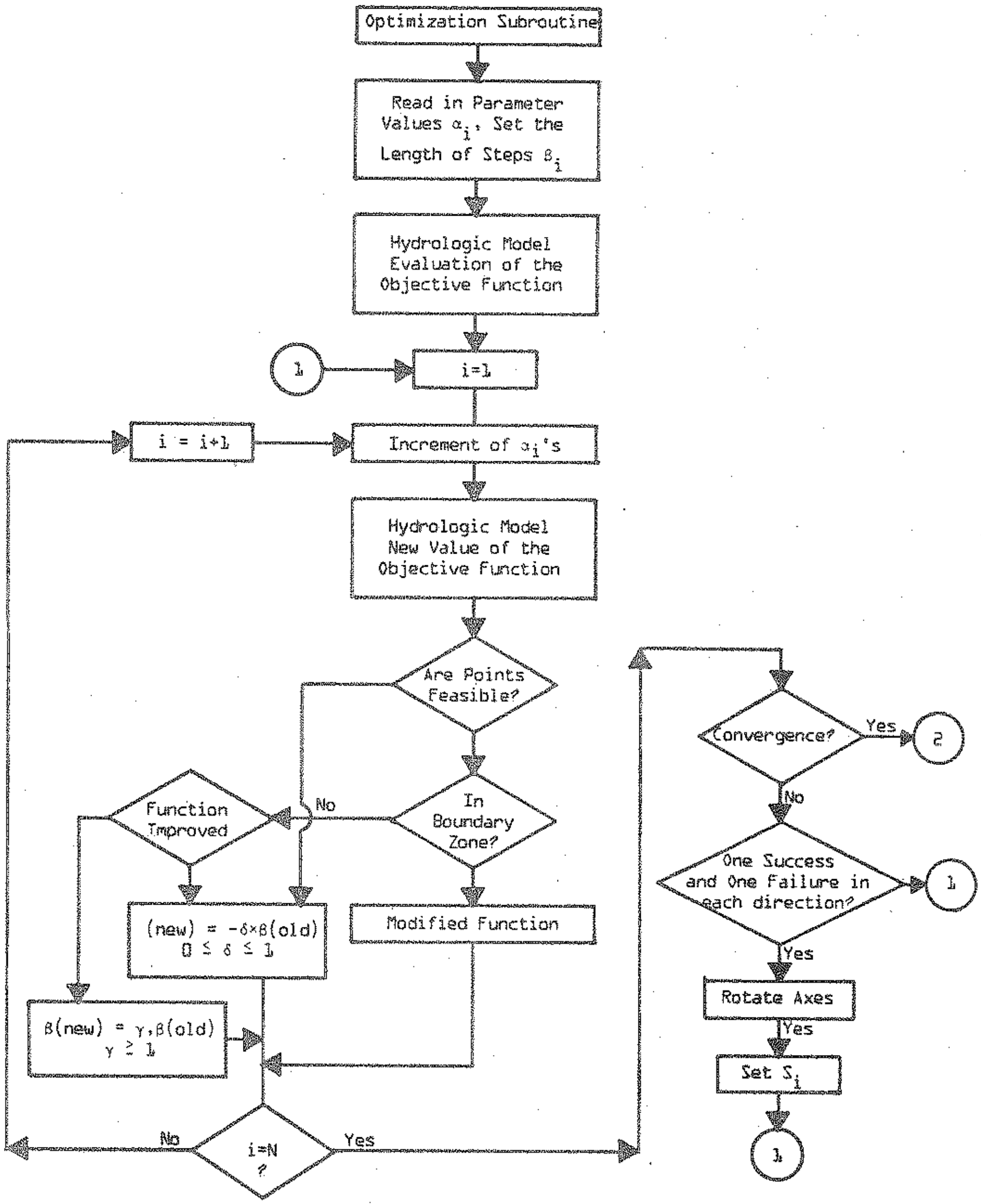


Figure 3.2 Computations in Rosenbrock's Method

The Box-Complex method, a constrained version of Box's Simplex method (Box (1965)), is a direct search method. The Complex method searches for the maximum value of the function (to find minimum, $-f$ is maximized) subject to M constraints of the form given in Eq. (3.26)

$$G_k \leqq \alpha_k \leqq H_k \qquad (3.26)$$
$$k=1,2,\cdots,M$$

The algorithm proceeds as follows (Kuester and Mize (1973)):

(1) An original "complex" of $k \geqq N+1$ points is generated. These consist of a feasible starting point x_0 and $k-1$ additional points generated from random numbers and constraints as shown in Eq. (3.27)

$$X_{i,j}=G_i+r_{i,j}(H_i-G_i) \qquad (3.27)$$
$$i=1,2,\cdots,N$$
$$j=1,2,\cdots,k-1$$

In Eq. (3.27) $r_{i,j}$ are uniformly distributed pseudo-random numbers over the interval (0,1). Box suggested $k=2N$.

(2) The selected points must satisfy both the explicit and implicit constraints. If at any time the explicit constraints are violated, the point is moved a small

distance δ inside the violated limit. If an implicit constraint is violated, the point is moved one half of the distance to the centroid of the remaining points.

$$X_{i,j}(new)=(X_{i,j}(old)+X^{+}_{i,q})/2.0 \tag{3.28}$$

where
$$X^{+}_{i,q}=\frac{1}{k-1}\left[\sum_{j=1}^{k}X_{i,j}-X_{i,j}(old)\right] \tag{3.29}$$

$$i=1,2,\cdots,N$$

This process is repeated as necessary until all the implicit constraints are satisfied.

(3) The objective function is evaluated at each point, and the point of lowest function values is replaced by a point as indicated by Eq. (3.30)

$$X_{i,j}(new)=\alpha[X^{+}_{i,q}-X_{i,j}(old)]+X^{+}_{i,q} \tag{3.30}$$
$$i=1,2,\cdots,N$$

Where α is a reflection coefficient. A value of 1.3 is recommended.

(4) If the point is the worst, it is moved half way towards the centroid, $X^{+}_{i,q}$ of the remaining points to get a new trial point. The new point is checked against the

constraints and is adjusted as before if the constraints are violated.

(5) The optimization scheme is assumed to have converged when objective function values at each point are within β units for γ consecutive iterations. β and γ are prespecified inputs to the program.

To be effective, the optimization method needs a concave objective function. Because of the complex structure of hydrological models such as ILLUDAS, six parameters of which are to be optimally estimated, it is not easy to prove the concavity of the objective function surface. Therefore, the optimal parameter estimated may not be optimal globally. However, methods given in Chapter IV may be used for checking whether the parameter estimates are locally or globally optimum.

3.7 Results

Split samples were used in the regeneration and prediction analyses. The first set of the data were used to estimate the optimal parameter estimates by the methods discussed earlier. The hydrographs computed by using the optimal parameters and the observed rainfall which was also used to estimate the optimal parameters are called optimized hydrographs. The optimized hydrographs are compared with the corresponding observed hydrographs. The observed hydrographs are also compared with uncalibrated hydrographs which are computed by using observed data and the parameter values which are selected on the basis of recommendations in user manuals. The advantages of optimal parameter estimation methods are brought out by these comparisons. This part of investigation is called the regeneration study.

The average or the "most probable" parameter values estimated in the regeneration study were then used along with the observed rainfall hyetographs which were not used in the regeneration study to predict runoff. These predicted hydrographs are compared with observed hydrogrpahs and the uncalibrated hydrographs. This part of the investigation is called the prediction study. Typical results from the regeneration and prediction studies are separately discussed below.

3.7.1 Regeneration Study Results From ILLUDAS

3.7.1.1 Upper Ross-Ade Watershed

The observed, optimized, and uncalibrated hydrographs for three storms in the Upper Ross watershed are shown in Fig. 3.3. The upper, initial, and lower values, for each of the six parameters which were optimally estimated are listed in Table 3.5. The optimal values of the six parameters, the arithmatic average of these parameters, the observed and computed peaks, times to peak, volumes, and their initial and final objective function values are given in Table 3.6.

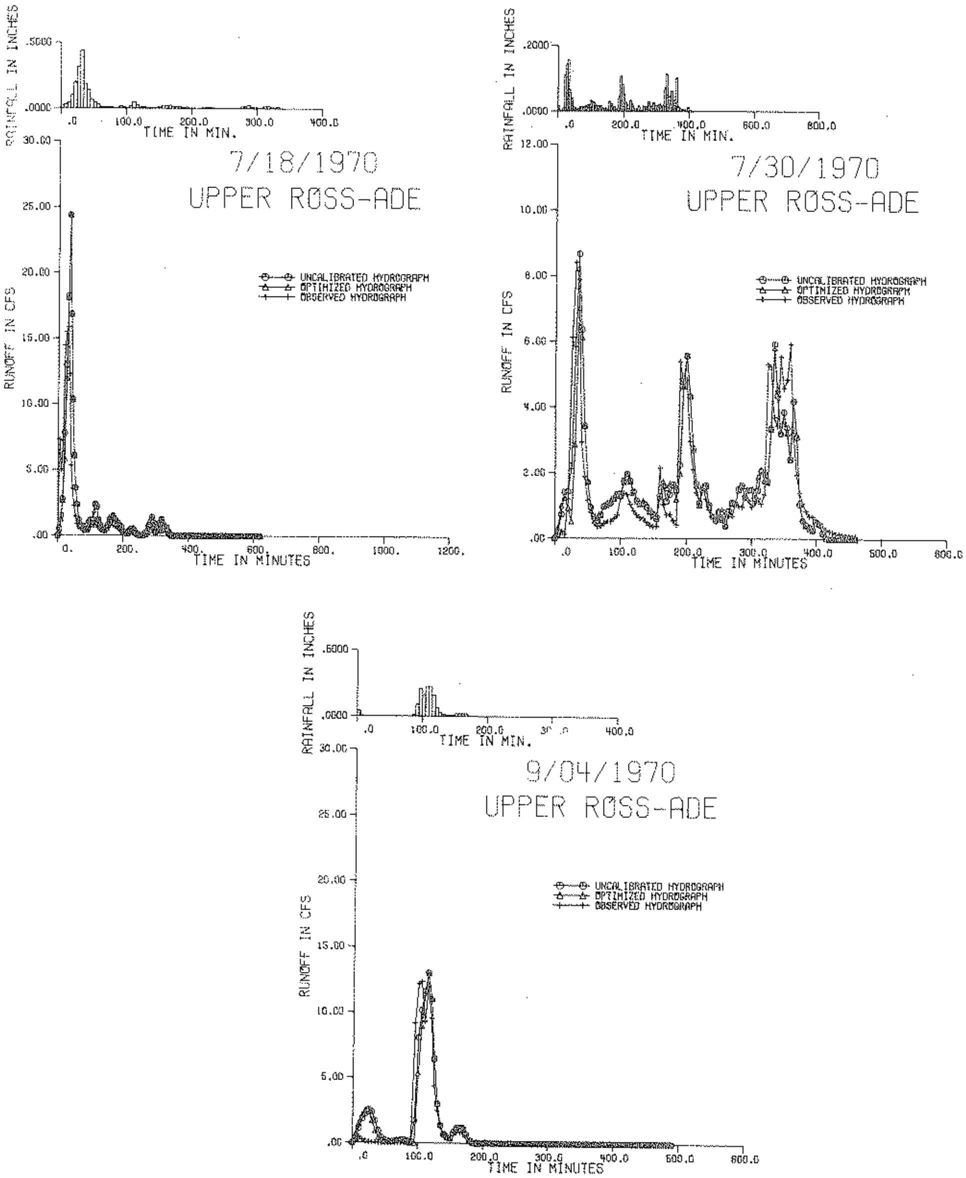


Figure 3.3 The Regenerated Hydrographs of ILLUDAS. (Upper Ross-Ade Watershed Data)

Table 3.5 The Lower, Initial, and Upper Parameter Values Used in ILLUDAS

	Parameter					
	ABSTRT	DEPG	FI	FO	FC	KIF
Lower	0.0	0.0	0.0	3.0	0.1	1.0
Initial	0.1	0.2	0.0	8.0	0.5	2.0
Upper	0.2	0.3	6.0	10.0	1.0	3.0

Table 3.6 Results of the Regeneration Study of ILLUDAS.
(Upper Ross-Ade Watershed Data)

STORM DATE	OPTIMIZED VALUES						OBSERVED			UNCALIBRATED HYDROGRAPHS				OPTIMIZED HYDROGRAPHS			
Mo/Day/Year	ABSTRT [in]	DEPG [in]	FI [in]	FO [in/hr]	FC [in/hr]	KIF [1/hr]	Peak [cfs]	Time to Peak [min]	Volume [in]	Peak [cfs]	Time to Peak [min]	Volume [in]	Objective Function Value	Peak [cfs]	Time to Peak [min]	Volume [in]	Objective Function Value
7/ 18 /1970	0.152	0.262	1.411	8.389	0.883	1.947	15.9	30	0.33	24.4	35	0.42	4.3×10^7	24.4	35	0.41	3.3×10^7
7/ 30 /1970	0.198	0.196	0	3.778	0.500	1.889	8.4	30	0.41	8.7	35	0.43	1.9×10^6	7.0	35	0.41	76
9/ 04 /1970	0.196	0.003	1.889	8.350	0.532	1.559	12.3	105	0.23	13.0	115	0.25	2.7×10^6	13.0	115	0.23	3.7×10^4
Arithmetic Average	0.182	0.154	1.100	6.839	0.638	1.798											

These results do not demonstrate the advantages of the optimal parameter estimation procedure. The initial parameter estimates are close to the final optimal estimates and hence the advantages of using the optimal parameter estimation method are not obvious in this case. However, the match between the observed and the optimized hydrographs is better than that between the observed and the uncalibrated hydrographs. The objective function values are smaller for the optimized hydrographs than for the uncalibrated hydrographs. The optimized hydrograph volumes are also slightly closer to observed volumes than the uncalibrated volumes.

3.7.1.2 Oakdale Avenue Basin

Typical observed, optimized, and uncalibrated hydrographs from the Oakdale Avenue Basin are shown in Fig. 3.4. The parameter estimates and other related information are summarized in Table 3.7. The same initial, upper, and lower parameter values listed in Table 3.5 were used here also. The volumes under the optimized hydrographs are closer to observed volumes in all cases. The peak flows are better reproduced by the optimized hydrographs than by the uncalibrated hydrographs. Time to peak values are similar in both cases. The superiority of optimized hydrographs over the uncalibrated hydrographs are clearly brought out by these results. The observed peak flows and volumes are plotted against the peak flows and volumes of optimized and

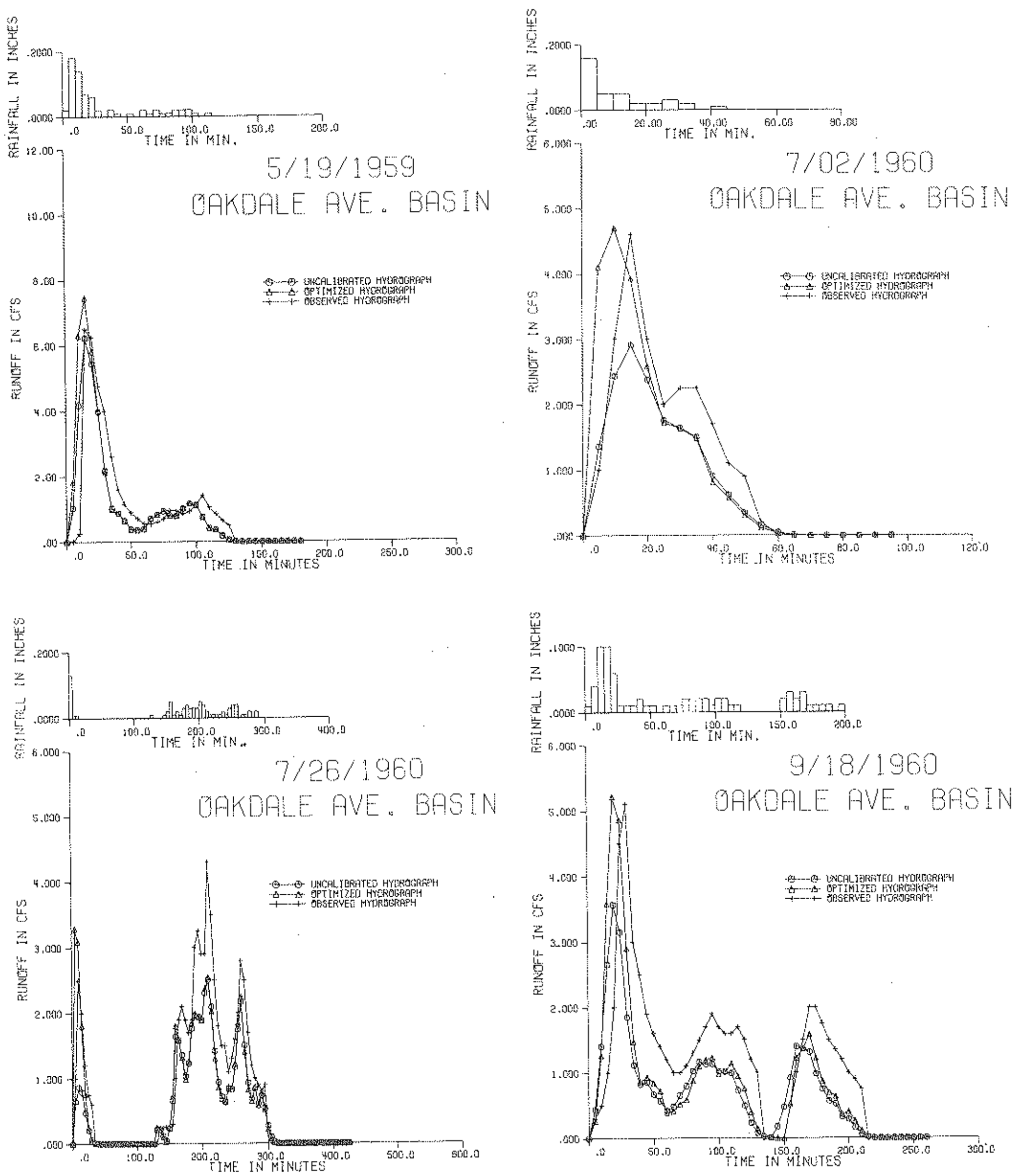


Figure 3.4 The Regenerated Hydrographs from ILLUDAS. (Oakdale Avenue Basin Data)

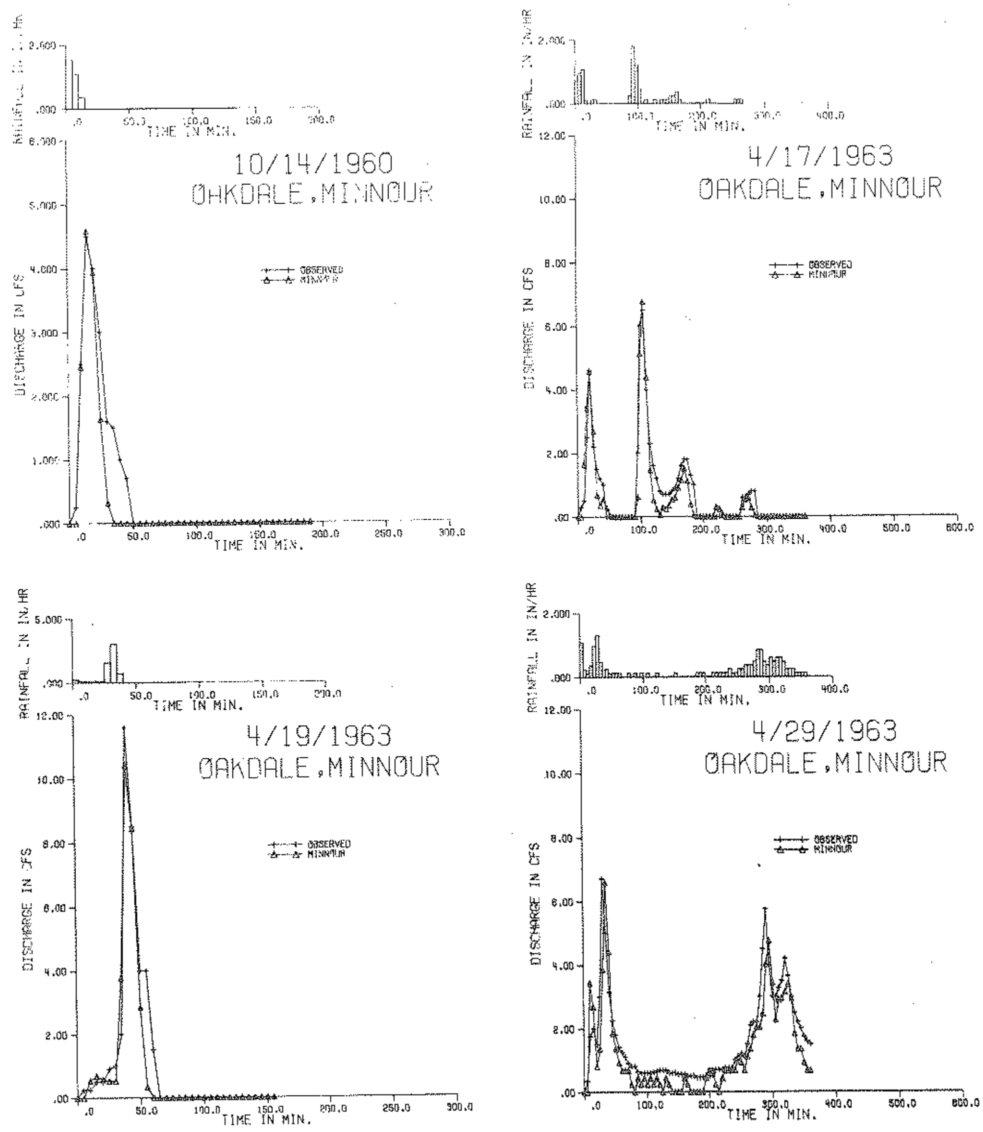


Figure 3.4, cont.

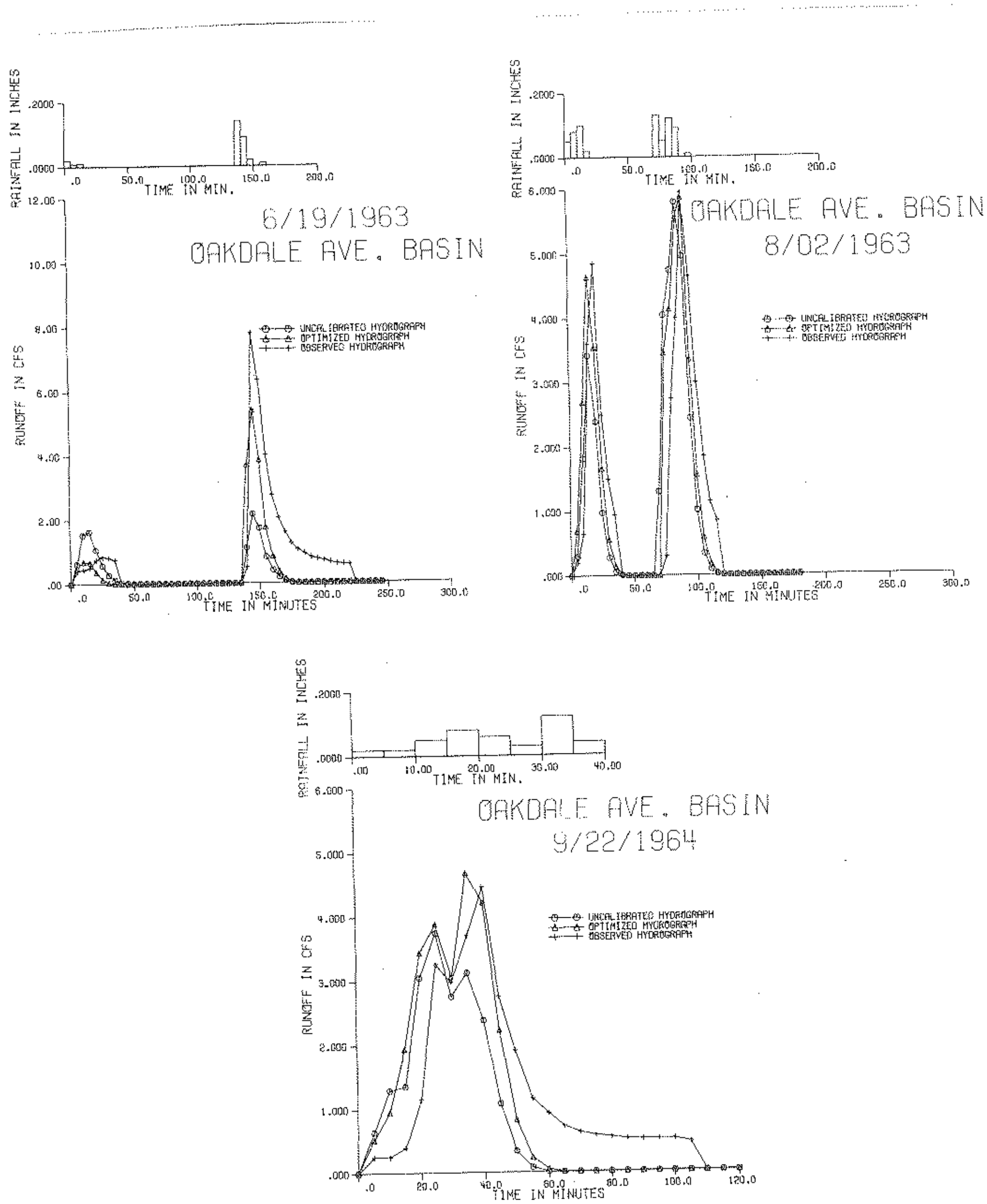


Figure 3.4, cont.

Table 3.7 Results of the Regeneration Study of ILLUDAS. (Oakdale Avenue Basin Data)

STORM DATE	OPTIMIZED VALUES						OBSERVED			UNCALIBRATED HYDROGRAPHS				OPTIMIZED HYDROGRAPHS			
Mo/Day/Year	ABSTRT [in]	DEPG [in]	FI [in]	FO [in/hr]	FC [in/hr]	KIF [1/hr]	Peak [cfs]	Time to Peak [min]	Volume [in]	Peak [cfs]	Time to Peak [min]	Volume [in]	Objective Function Value	Peak [cfs]	Time to Peak [min]	Volume [in]	Objective Function Value
5/19/1959	0.025	0.050	0	7.933	0.279	2.407	6.5	15	0.26	6.2	15	0.23	9.7×10^5	7.5	15	0.26	27
7/02/1960	0.005	0.300	0	8.100	0.600	2.100	4.6	15	0.14	2.9	15	0.10	1.6×10^6	4.7	10	0.14	8
7/26/1960	0	0.050	0	7.933	0.279	2.407	4.3	210	0.42	2.5	210	0.27	2.2×10^7	2.5	210	0.31	1.1×10^7
9/18/1960	0	0.050	0	7.933	0.279	2.407	5.1	30	0.39	3.6	20	0.24	2.3×10^7	5.2	20	0.28	1.2×10^7
10/14/1960	0	0.050	0	7.933	0.279	2.407	4.5	15	0.12	3.0	15	0.06	4.3×10^5	5.2	10	0.10	5.7×10^5
4/17/1963	0	0.050	0	7.933	0.279	2.407	6.5	105	0.35	5.8	100	0.25	9.6×10^6	6.1	100	0.29	3.2×10^6
4/19/1963	0	0.195	4.004	8.016	0.497	2.000	11.6	40	0.22	5.2	35	0.16	4.7×10^6	9.1	35	0.22	355
4/29,30/1963	0	0.050	0	7.933	0.279	2.407	6.7	30	0.75	4.9	30	0.	7.0×10^7	5.0	30	0.54	5.0×10^7
6/19/1963	0	0.050	0	7.933	0.279	2.407	7.8	145	0.24	2.2	145	0.08	2.8×10^7	5.4	145	0.12	1.6×10^7
8/02/1963	0.024	0.029	0.295	7.869	0.299	2.147	5.9	90	0.25	5.8	85	0.22	9.9×10^6	5.9	90	0.25	39
9/22/1964	0	0.050	0	7.933	0.279	2.407	4.4	40	0.18	3.7	25	0.13	3.1×10^6	4.7	35	0.17	1.9×10^5
Majority Optimal Values	0	0.050	0	7.933	0.279	2.407											
Arithmetic Average	0.005	0.084	0.391	7.950	0.330	2.318											

uncalibrated hydrographs in Figs. 3.5 and 3.6. Some of the related statistics which are defined below in Eqs. (3.31)-(3.33) are given in Table 3.8.

Table 3.8 Statistics Related to Optimized and Uncalibrated Peak Flows and Volumes Computed by ILLUDAS and Observed Data

	Peak Flows			Volumes		
	R_0	R	ISE	R_0	R	ISE
Optimized	0.96	0.81	6.57	0.93	0.96	8.87
Uncalibrated	0.81	0.39	13.76	0.88	0.95	11.90

R_0: Special Correlation Coefficient

$$R_0 = \frac{2 \sum_{i=1}^{N} X_i Y_i - \sum_{i=1}^{N} Y_i^2}{\sum_{i=1}^{N} X_i^2} \tag{3.31}$$

R: Correlation Coefficient

$$R=\frac{N\sum_{i=1}^{N} X_iY_i-(\sum_{i=1}^{N} X_i)(\sum_{i=1}^{N} Y_i)}{\{[N\sum_{i=1}^{N} X_i^2-(\sum_{i=1}^{N} X_i)^2][N\sum_{i=1}^{N} Y_i^2-(\sum_{i=1}^{N} Y_i)^2]\}^{0.5}} \qquad (3.32)$$

ISE: Integral Square Error

$$ISE=\frac{[\sum_{i=1}^{N} (X_i-Y_i)^2]^{0.5}}{\sum_{i=1}^{N} X_i}\times 100 \qquad (3.33)$$

where X_i: observed values

Y_i: computed values

The results shown in Figs. 3.5 and 3.6 and the associated statistics show that in general, the optimized hydrographs have peak flows which are closer to observed peak flows. Although the volumes under the optimized hydrographs are closer to the observed volumes than the volumes given by uncalibrated hydrographs, the improvement is not as striking as for the peak flows.

The statistical parameters clearly show that both the optimized hydrograph peak flows and volumes are superior to those of uncalibrated hydrographs. The improvement brought about by the optimal parameters in the regeneration of hydrographs is clearly seen by these results.

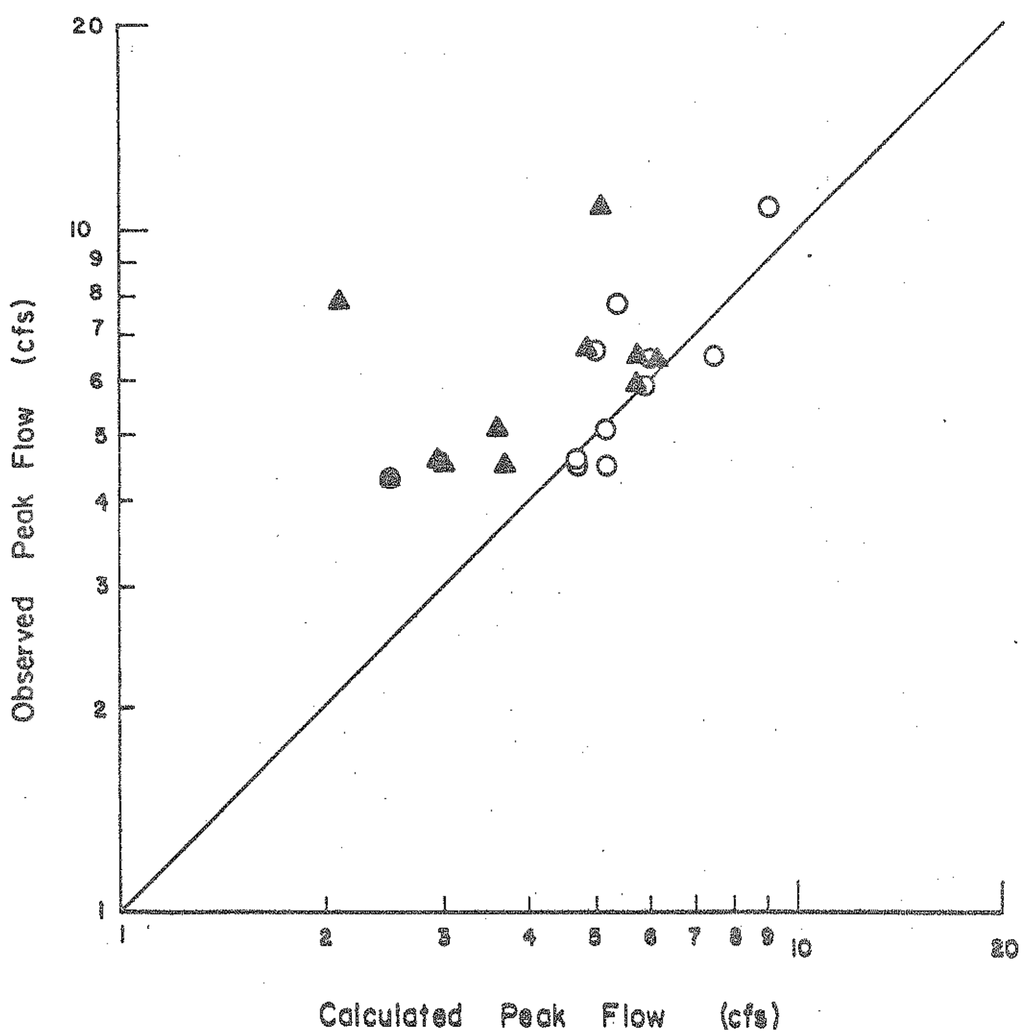


Figure 3.5 The Comparison of the Uncalibrated (Triangles) and Optimized (Circles) Peak Flows. (ILLUDAS, Oakdale Avenue Basin Data)

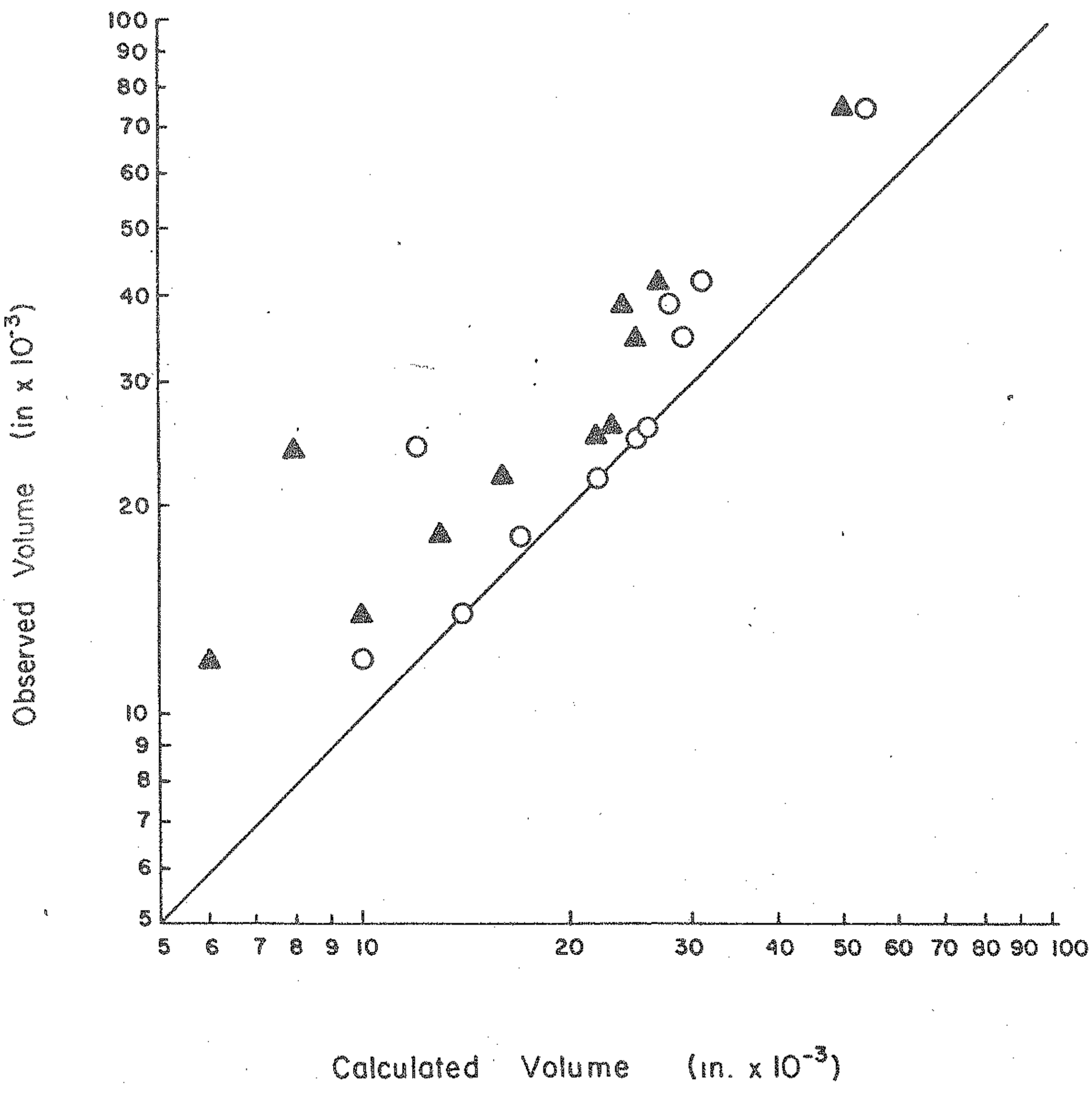


Figure 3.6 The Comparison of the Uncalibrated (Triangles) and Optimized (Circles) Volumes. (ILLUDAS, Oakdale Avenue Basin Data)

3.7.2 Regeneration Study Results From SWMM

3.7.2.1 Upper Ross-Ade Watershed

Typical observed, optimized, and uncalibrated hydrographs for Upper Ross-Ade watershed obtained by using the SWMM model are shown in Fig. 3.7. The initial estimates and the upper and lower bounds for each of the five parameters which were optimized are listed in Table 3.9. The optimal values of these parameters are given in Table 3.10. Hydrographs generated by using optimal parameter estimates match the observed hydrographs better and the improvement in the fit between observed and optimized hydrographs is indicated by slightly smaller values of objective function.

3.7.2.2 Oakdale Avenue Basin

Typical observed, optimized, and uncalibrated hydrographs from the Oakdale Avenue Basin are shown in Fig. 3.8 and the results are summarized in Table 3.11. For all the storms, peak flows, times to peak, and total volumes are better reproduced by hydrographs computed by using the optimal parameter estimates than by hydrographs computed by using the values suggested in the user's manual. The observed peak flows and volumes are plotted against the corresponding values computed by using the optimal and user's manual suggested values in Figs. 3.9 and 3.10. The superiority of the optimal hydrographs over the uncalibrated hydrographs is clearly demonstrated by these results. The statistical

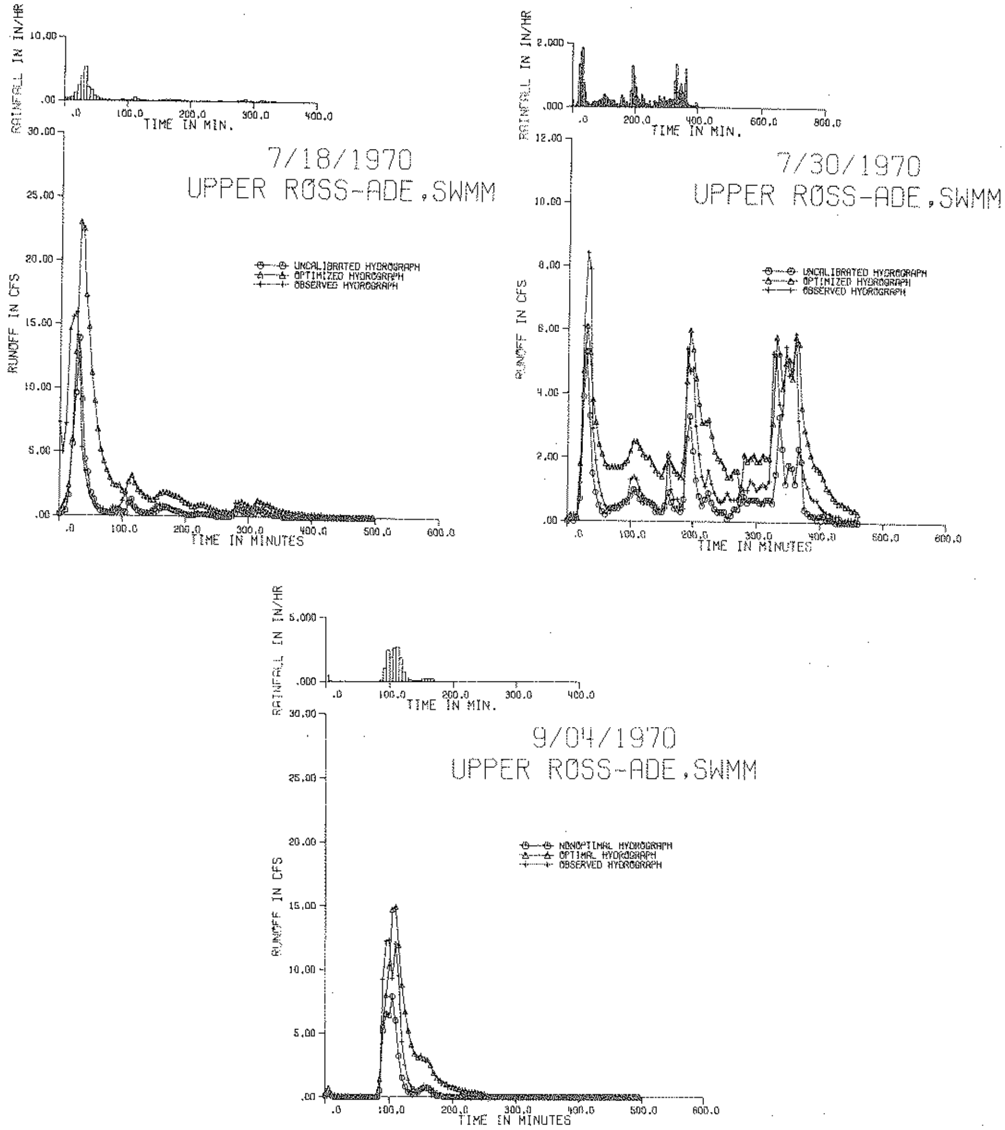


Figure 3.7 The Regenerated Hydrographs from SWMM. (Upper Ross-Ade Watershed Data)

Table 3.9 The Lower, Initial, and Upper Bound Values of the SWMM Parameters

Notation Used in SWMM	Description	Lower Bounds	Initial Estimates	Upper Bounds
WW(7)	Impervious storage (inches)	0	0.10	0.20
WW(8)	Pervious storage (inches)	0	0.20	0.30
WW(9)	Max. Infil. Rate (in/hr)	3.00	8.00	10.00
WW(10)	Min. Infil. Rate (in/hr)	0.10	0.50	1.00
WW(11)	Decay Constant(1/sec)	0.00028	0.00053	0.00083

Table 3.10 Results of the Regeneration Study of SWMM. (Upper Ross-Ade Watershed Data)

STORM DATE	OPTIMIZED VALUES					OBSERVED			UNCALIBRATED HYDROGRAPHS				OPTIMIZED HYDROGRAPHS			
	WW(7)	WW(8)	WW(9)	WW(10)	WW(11)	Peak	Time to Peak	Volume	Peak	Time to Peak	Volume	Objective Function Value	Peak	Time to Peak	Volume	Objective Function Value
Mo/Day/Year	[in]	[in]	[in/hr]	[in/hr]	[sec^{-1}]	[cfs]	[min]	[in]	[cfs]	[min]	[in]		[cfs]	[min]	[in]	
7/18/1970	0.192	0.297	8.979	0.900	0.00065	15.9	30	0.33	13.9	35	0.20	5.2×10^8	23.0	35	0.62	4.5×10^8
7/30/1970	0.192	0.297	8.979	0.900	0.00065	8.4	30	0.41	5.3	35	0.20	2.9×10^8	6.1	35	0.61	2.4×10^8
9/04/1970	0.192	0.297	8.979	0.900	0.00065	12.3	105	0.23	7.9	110	0.12	1.3×10^8	14.9	115	0.36	9.9×10^7
Final Value	0.192	0.297	8.979	0.900	0.00065											

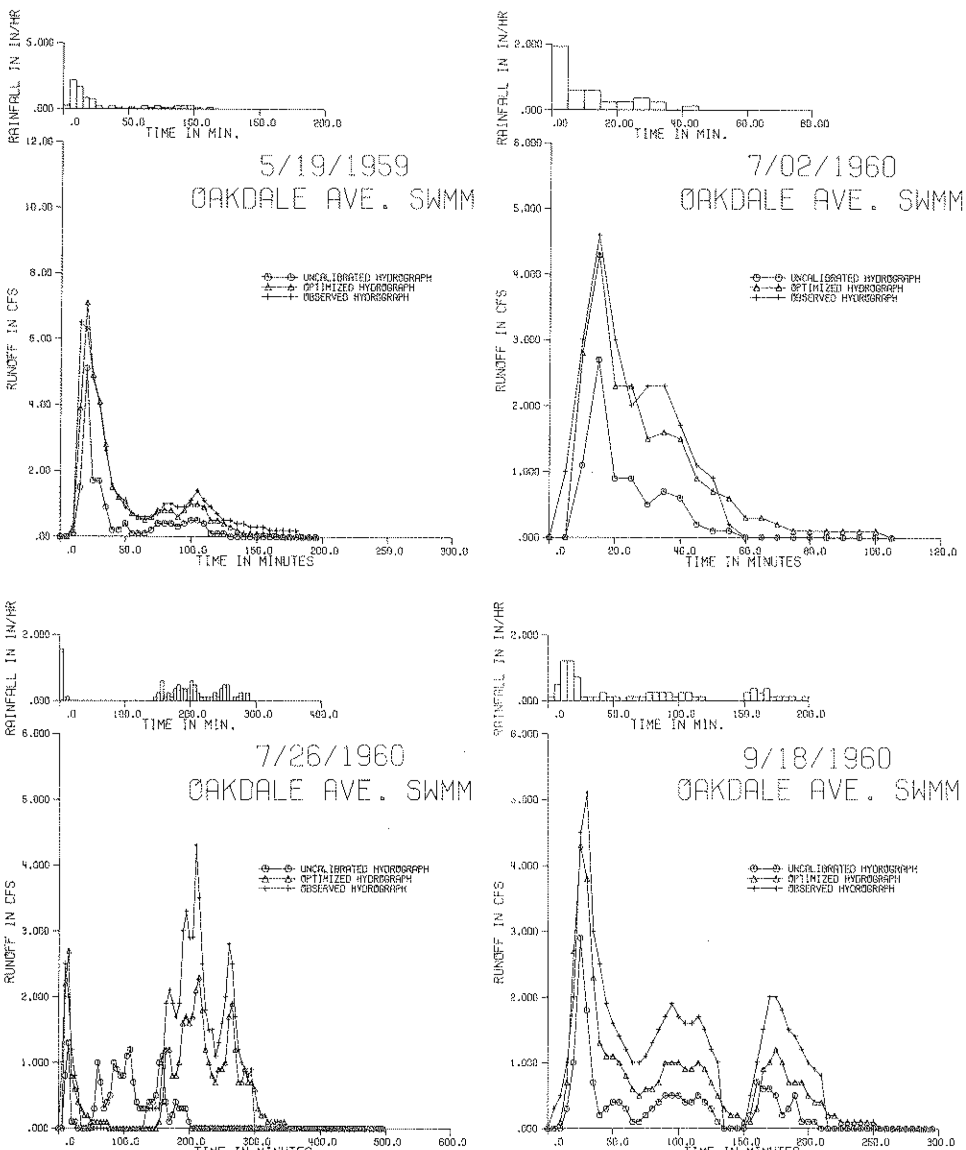


Figure 3.8 The Regenerated Hydrographs from SWMM. (Oakdale Avenue Basin Data)

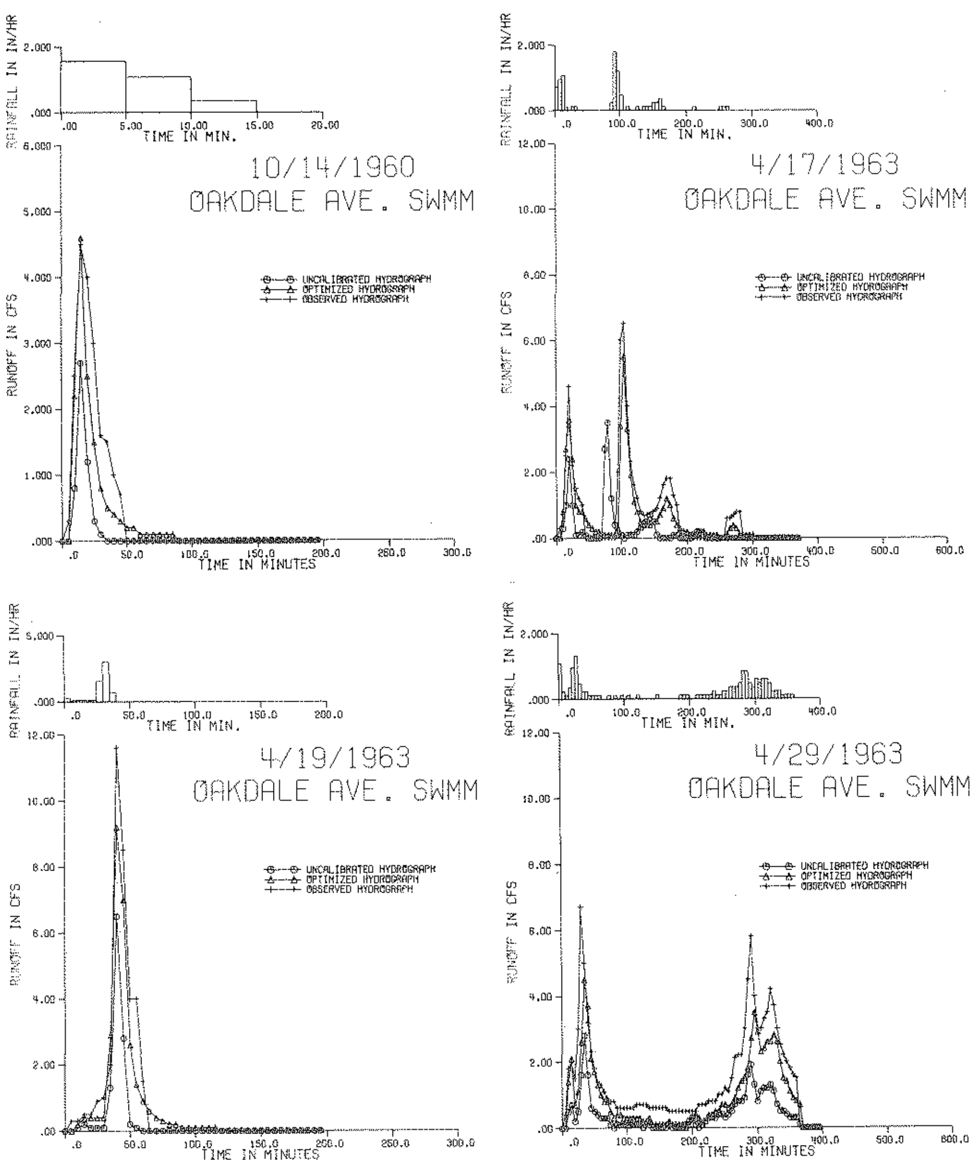


Figure 3.8, cont.

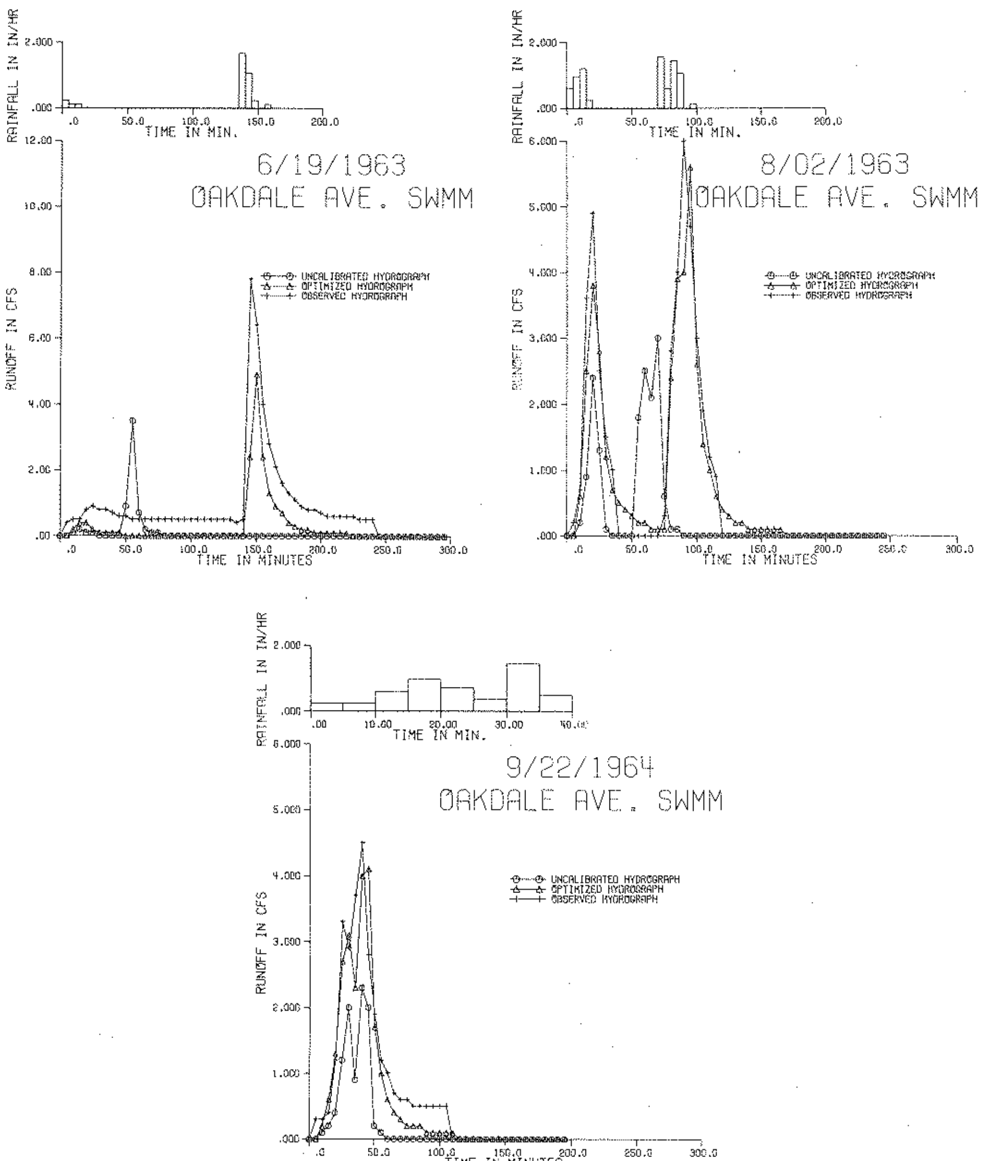


Figure 3.8, cont.

Table 3.11 Results of the Regeneration Study of SWMM. (Oakdale Avenue Basin Data)

STORM DATE	OPTIMIZED VALUES					OBSERVED			UNCALIBRATED HYDROGRAPHS				OPTIMIZED HYDROGRAPHS			
	WW(7)	WW(8)	WW(9)	WW(10)	WW(11)	Peak	Time to Peak	Volume	Peak	Time to Peak	Volume	Objective Function Value	Peak	Time to Peak	Volume	Objective Function Value
Mo/Day/Year	[in]	[in]	[in/hr]	[in/hr]	[sec^{-1}]	[cfs]	[min]	[in]	[cfs]	[min]	[in]		[cfs]	[min]	[in]	
5/19/1959	0	0	8.00	0.60	0.00055	6.5	15	0.26	5.1	20	0.10	8.44×10^7	7.1	20	0.25	8.37×10^7
7/02/1960	0	0	8.00	0.60	0.00055	4.6	15	0.14	2.7	15	0.05	2.16×10^7	4.3	15	0.13	2.14×10^7
7/26/1960	0	0	8.00	0.60	0.00055	4.3	210	0.42	1.2	100	0.12	1.88×10^8	2.4	215	0.28	1.86×10^8
9/18/1960	0	0	8.00	0.60	0.00055	5.1	30	0.39	2.9	25	0.11	1.63×10^8	4.3	25	0.25	1.61×10^8
10/14/1960	0	0	8.00	0.60	0.00055	4.5	15	0.12	2.7	15	0.03	1.63×10^7	4.6	15	0.09	1.61×10^7
4/17/1963	0	0	8.00	0.60	0.00055	6.5	105	0.35	3.5	80	0.11	1.31×10^8	5.5	105	0.27	1.3×10^8
4/19/1963	0	0	8.00	0.60	0.00055	11.6	40	0.22	6.5	40	0.07	5.5×10^7	9.2	40	0.18	5.4×10^7
4/29,30/1963	0	0	8.00	0.60	0.00055	6.7	30	0.75	2.8	35	0.21	6.1×10^8	4.5	35	0.48	6.09×10^8
6/19/1963	0	0	8.00	0.60	0.00055	7.8	145	0.24	3.5	55	0.04	1.1×10^8	4.9	150	0.10	$1. \times 10^8$
8/02/1963	0	0	8.00	0.60	0.00055	5.9	90	0.25	3.0	70	0.10	6.7×10^7	5.6	95	0.23	6.65×10^7
9/22/1964	0	0	8.00	0.60	0.00055	4.4	40	0.18	2.0	30	0.06	3.5×10^7	4.1	45	0.15	3.49×10^7
Final Value	0	0	8.00	0.60	0.00055											

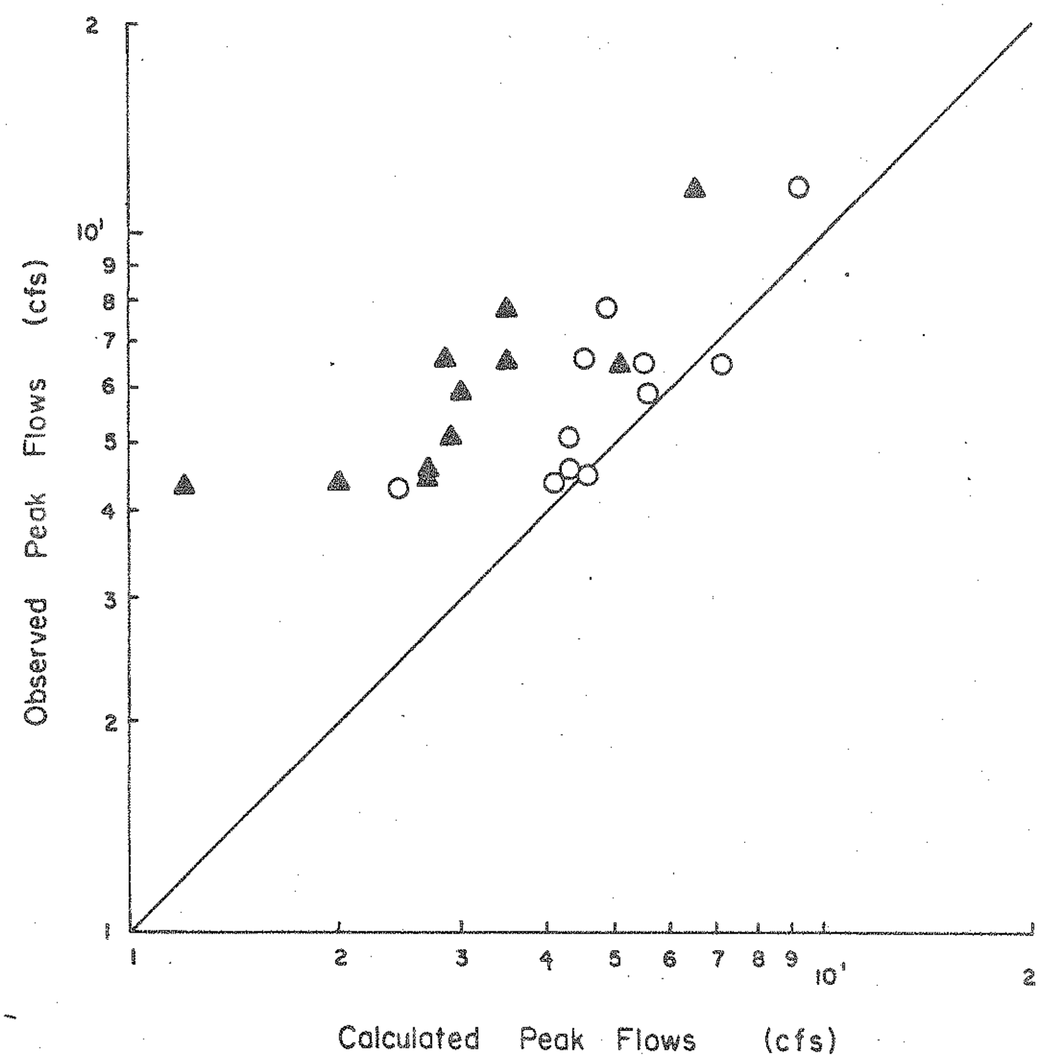


Figure 3.9 The Comparison of the Uncalibrated (Triangles) and Optimized (Circles) Peak Flows. (SWMM, Oakdale Avenue Basin Data)

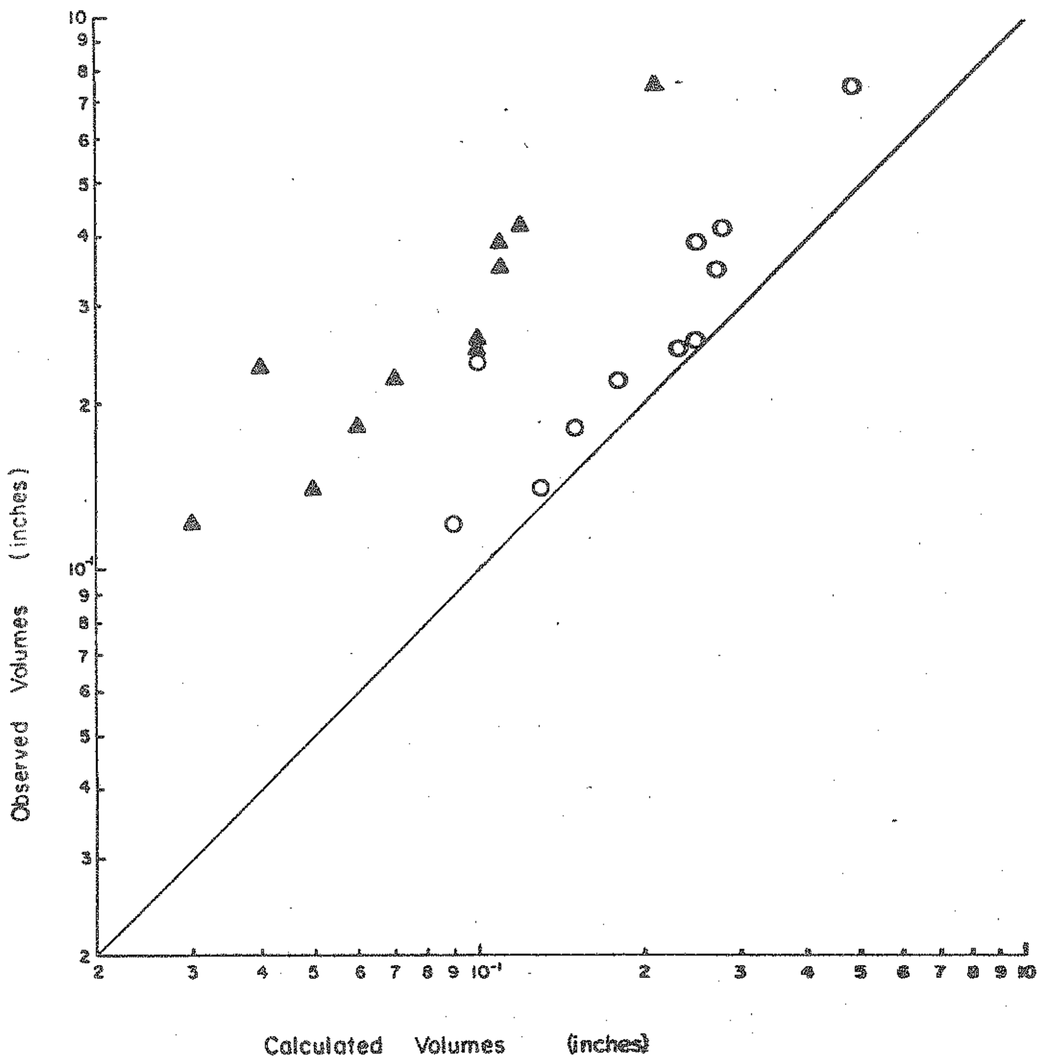


Figure 3.10 The Comparison of the Uncalibrated (Triangles) and Optimized (Circles) Volumes. (SWMM, Oakdale Avenue Basin Data)

parameters R_0, R, and ISE which are defined before are also calculated and presented in Table 3.12. They also demonstrate the superiority of the optimal parameter estimates.

3.7.3 Regeneration Study Results From MINNOUR

The MINNOUR (Chu (1978)) was developed and tested by using a few storms from the Oakdale Avenue Basin. The regeneration performance of the MINNOUR model was tested in this study and was compared with ILLUDAS and SWMM. The parameter limits and other information are kept the same or as close to each other as possible in all models. The initial values and the lower and upper limits of the parameter estimates are given in Table 3.13.

The objective function minimized in MINNOUR is the sum of squared deviations given in Eq. (3.34),

$$\text{Objective function} = \sum_{i=1}^{N} (O_i - C_i)^2 \tag{3.34}$$

where O_i: observed hydrograph ordinate at ith interval

C_i: calculated hydrograph ordinate at ith interval

N : total number of ordinates

Table 3.12 Statistics Related to Optimized and Uncalibrated Peak Flows and Volumes Computed by SWMM and Observed Data

	Peak Flows			Volumes		
	R_0	R	ISE	R_0	R	ISE
Optimized	0.95	0.85	7.35	0.89	0.94	11.35
Uncalibrated	0.77	0.87	15.17	0.50	0.95	24.45

Table 3.13 The Lower, Initial, and Upper Parameter Values in MINNOUR

	Lower Limit	Initial Estimates	Upper Limit
SDEPTH (inches)	3.0	3.6	12.0
SMI (%)	25.0	30.2	50.0
DSPERV (inches)	0.0	0.20	0.30
DSIMPV (inches)	0.0	0.10	0.20
TC (min.)	5.0	9.1	10.0

The modified Holtan's infiltration equation and the SCS hydrograph method were used in MINNOUR in this study.

The optimal parameter estimates in MINNOUR for the Oakdale Avenue Basin along with some of the information about observed and computed hydrographs are listed in Table 3.14. Typical observed and calculated hydrographs are plotted in Fig. 3.11. The sum of the squared deviations and the special correlation coefficient for each storm are summarized in Table 3.15. Calculated peak flows and volumes are plotted against the observed values in Figs. 3.12 and 3.13. The values of the special correlation coefficient, correlation coefficient, and Integral Square Error for the calculated peak flows and volumes against the observed data are listed as Table 3.16.

The results from MINNOUR are excellent for data from Oakdale Avenue Basin.

3.7.4 Prediction Study Results From ILLUDAS

Storms which were not used in the regeneration study were used in this part of the investigation. Two sets of optimal parameter estimates may be used in the prediction study. The first of these is the set of parameters which remained the same for most of the storms during regeneration study and these are called the "most probable" parameter values. The second set is simply the arithmetic average of the parameters estimated by using different data sets in the regeneration study.

Table 3.14 Results of the Regeneration Study of MINNOUR. (Oakdale Avenue Basin Data)

Storm Date	Optimal Parameter Values					Observed			Optimized		
M/D/YR	SDEPTH	SMI	DSPERV	DSIMPV	TC	Peak (cfs)	Time to Peak (min)	Volume (in)	Peak (cfs)	Time to Peak (min)	Volume (in)
5/19/1959	9.1	28.1	0.2822	0.0739	7.8	6.5	15	0.26	7.4	20	0.23
7/02/1960	3.6	30.2	0.2000	0.1000	9.1	4.6	15	0.14	4.4	15	0.12
7/26/1960	4.6	43.8	0.0742	0.1718	7.3	4.3	210	0.42	2.6	215	0.31
9/18/1960	4.6	43.8	0.0742	0.1718	7.3	5.1	30	0.39	5.5	25	0.27
10/14/1960	3.6	30.2	0.2000	0.1000	9.1	4.5	15	0.12	4.6	15	0.08
4/17/1963	10.9	31.7	0.2792	0.1800	6.8	6.5	105	0.35	6.8	105	0.28
4/19/1963	10.9	31.7	0.2792	0.1800	6.8	11.6	40	0.22	10.4	40	0.18
4/29/1963	12.0	31.4	0.2990	0.1141	5.0	6.7	30	0.75	6.6	35	0.57
6/19/1963	4.3	25.0	0.0010	0.0010	5.7	7.8	145	0.24	7.6	150	0.14
8/02/1963	5.5	36.9	0.1774	0.0454	8.1	5.9	90	0.25	5.2	95	0.23
9/22/1964	3.6	30.2	0.2000	0.1000	9.1	4.4	40	0.18	3.9	45	0.14

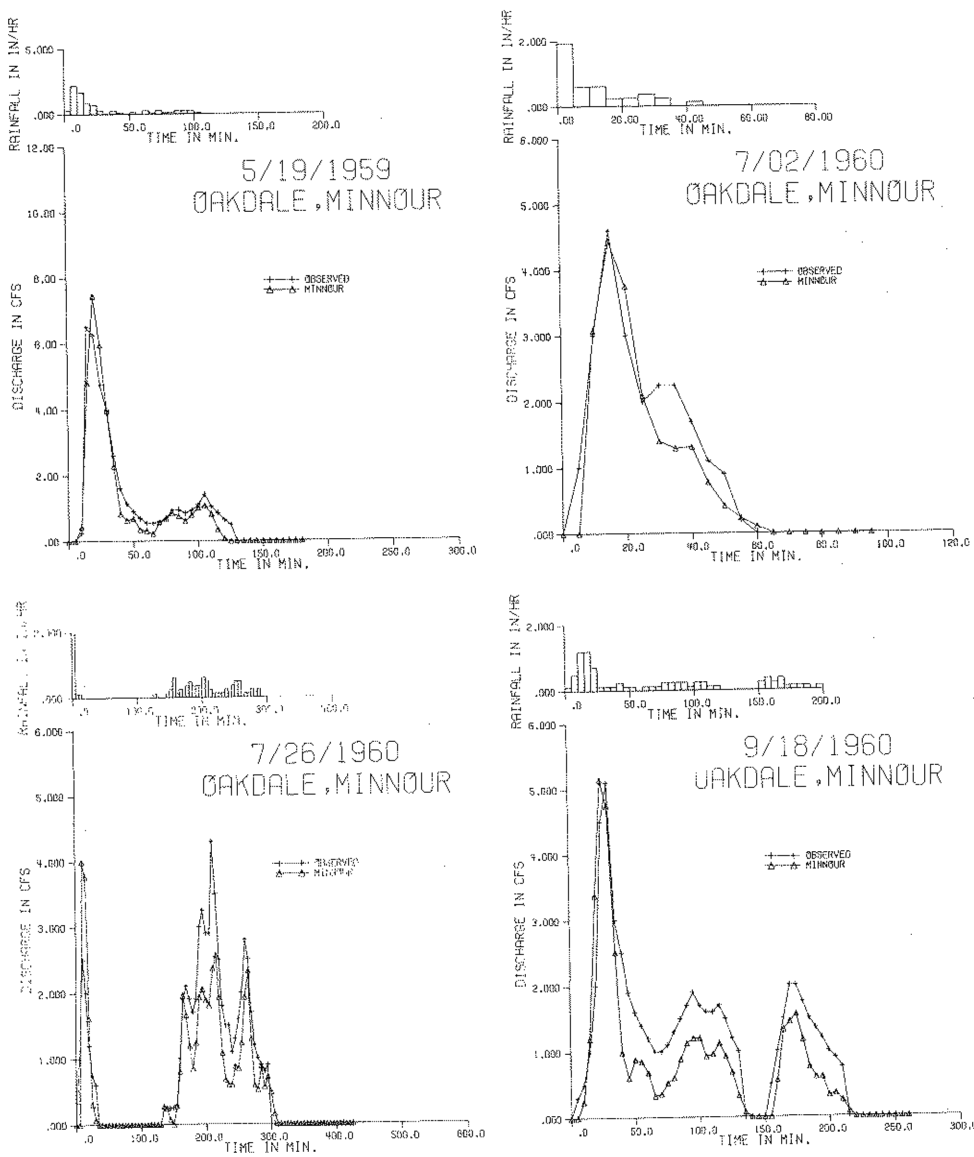


Figure 3.11 The Regenerated Hydrographs from MINNOUR. (Oakdale Avenue Basin Data)

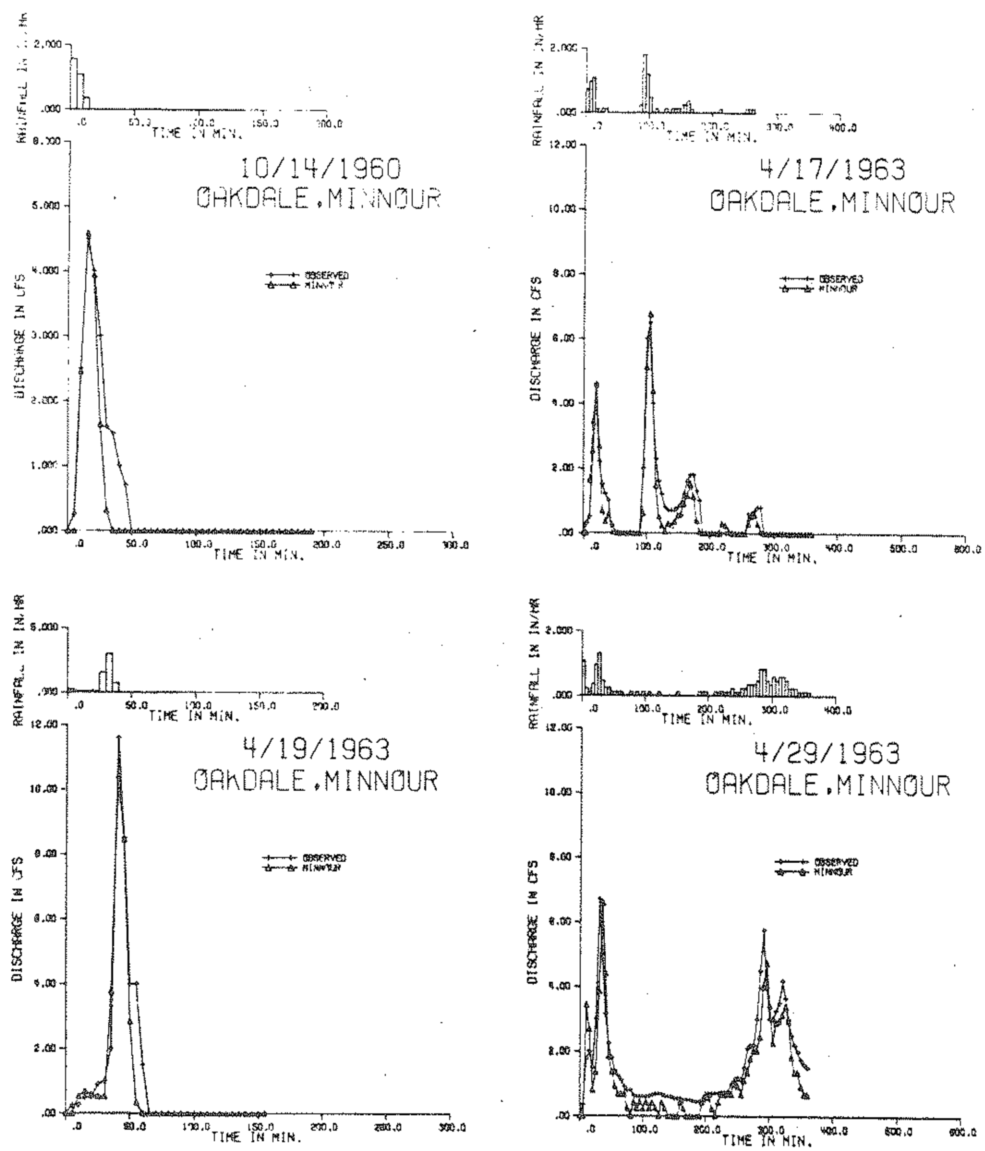


Figure 3.11, cont.

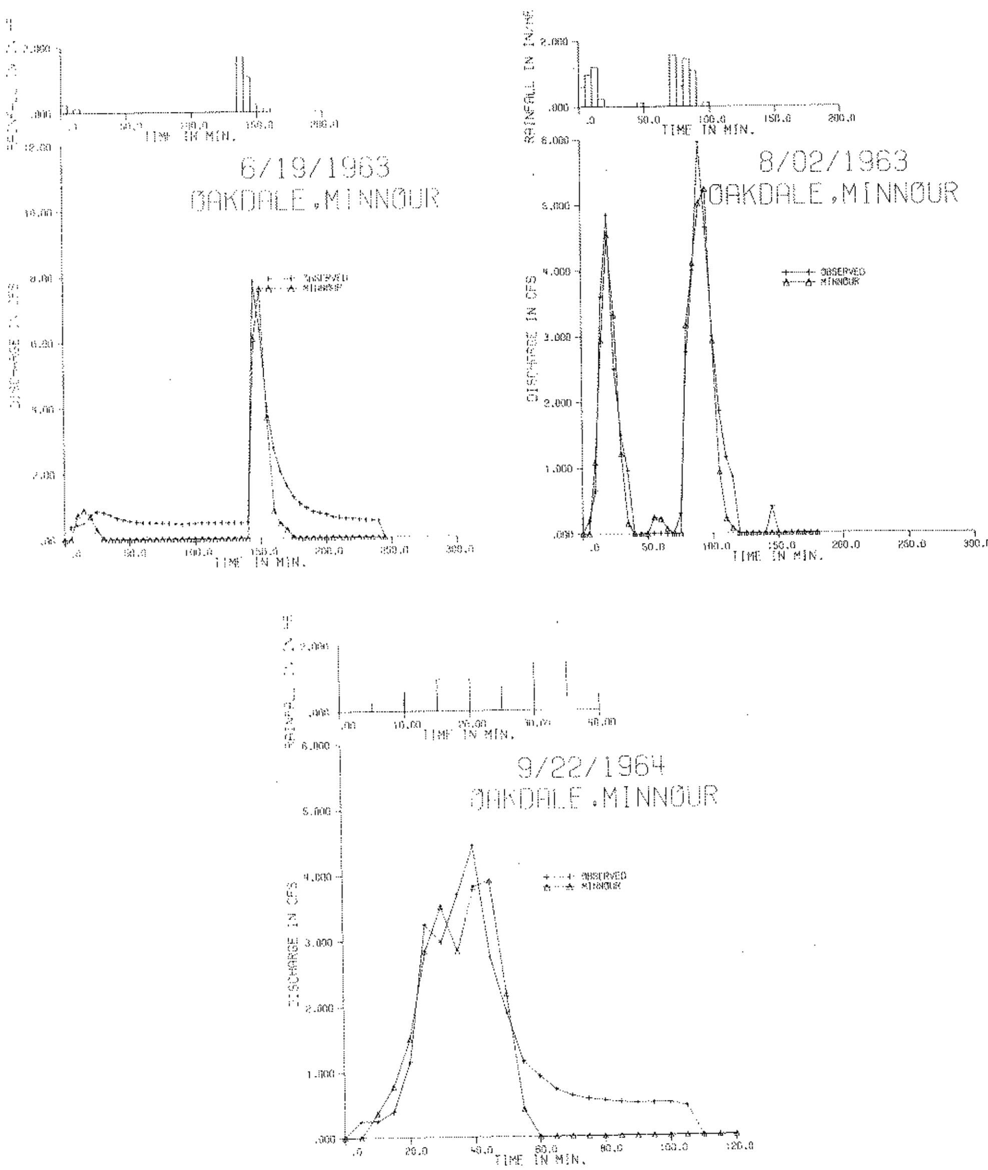


Figure 3.11, cont.

Table 3.15 The Sum of Squared Deviations and R_0 Values of MINNOUR

Storm Date M/D/YR	Sum of Squared Deviations	R_0
5/19/1959	8.17	0.9426
7/02/1960	3.73	0.9371
7/26/1960	24.20	0.8335
9/18/1960	16.98	0.8675
10/14/1960	7.32	0.8736
4/17/1963	14.63	0.9108
4/19/1963	22.29	0.9099
4/29/1963	39.62	0.8791
6/19/1963	27.53	0.8135
8/02/1963	6.21	0.9562
9/22/1964	7.50	0.8938

Table 3.16 The Statistics of Results from MINNOUR

	Peak Flows	Volumes
R_0	0.99	0.94
R	0.95	0.98
ISE	3.63	8.54

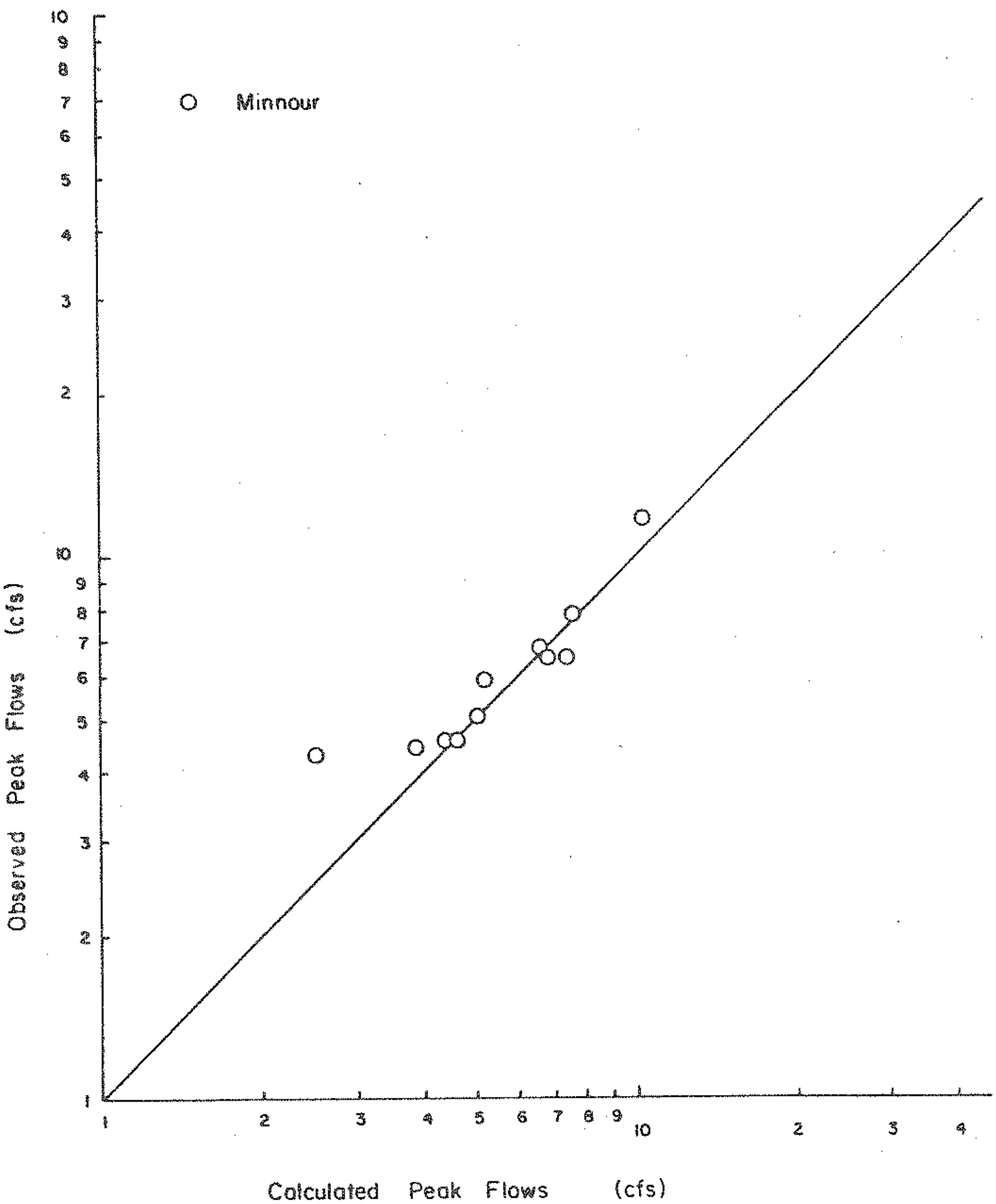


Figure 3.12 The Generated and Observed Peak Flows.
(MINNOUR, Oakdale Avenue Basin Data)

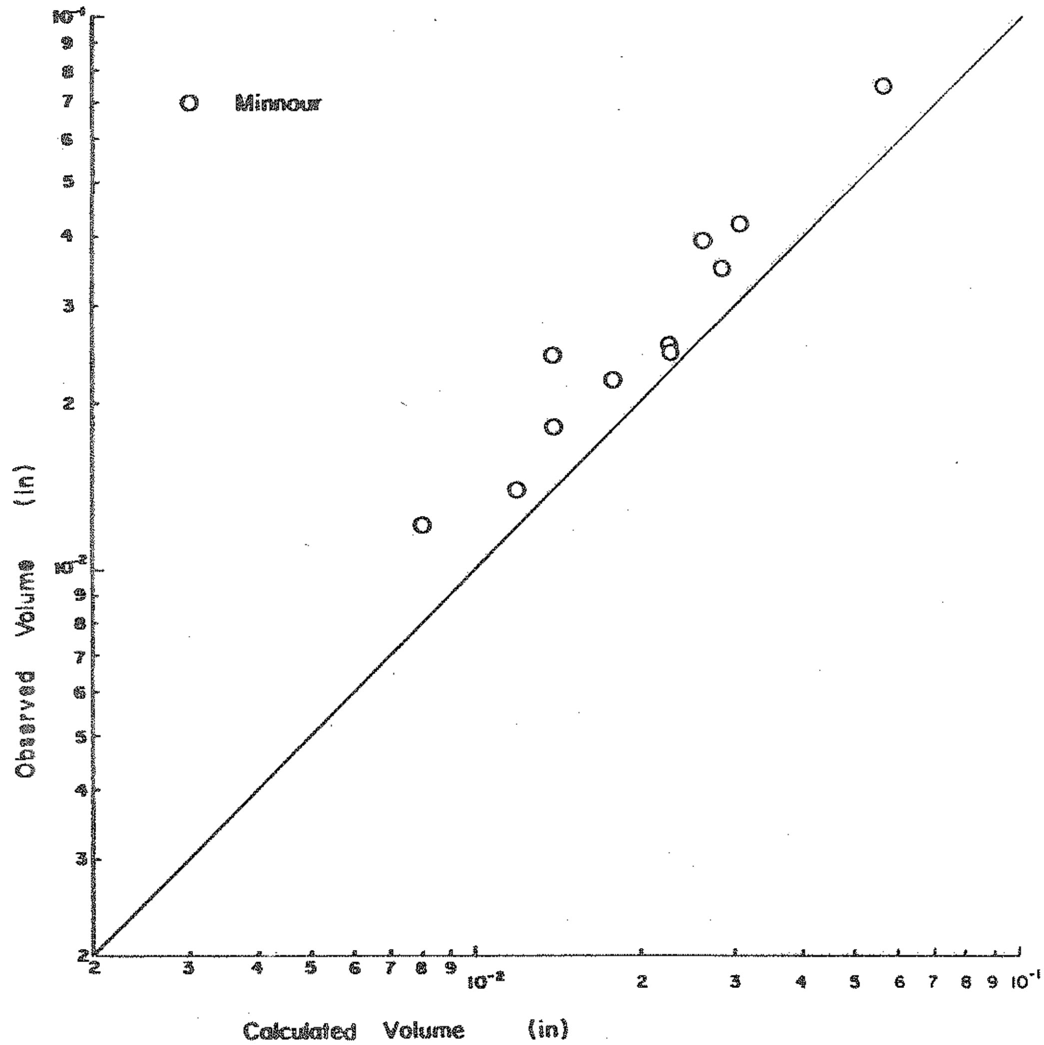


Figure 3.13 The Generated and Observed Volumes. (MINNOUR, Oakdale Avenue Basin Data)

3.7.4.1 Upper Ross-Ade Watershed

The arithmatic averages of the optimal parameter estimates were used as the optimal estimates of each of the parameters, as recommended by Liou (1970), to predict the model performance. A typical predicted and observed hydrograph is presented in Fig. 3.14. Some of the more important hydrograph characteristics and statistics used for comparison of observed and predicted hydrographs are given in Table 3.17. The statistics, R_0, R, and ISE, show that the optimized hydrographs match the observed hydrographs better than the uncalibrated hydrographs. The smaller peak on the optimized hydrograph may be partly explanined by the form of the objective function which was selected to emphasize good reproduction of both runoff rates and volumes. Nevertheless, the statistics between the observed and calculated hydrographs indicate that hydrographs estimated by using optimal parameter estimates show better agreement with observed hydrographs than the uncalibrated hydrographs.

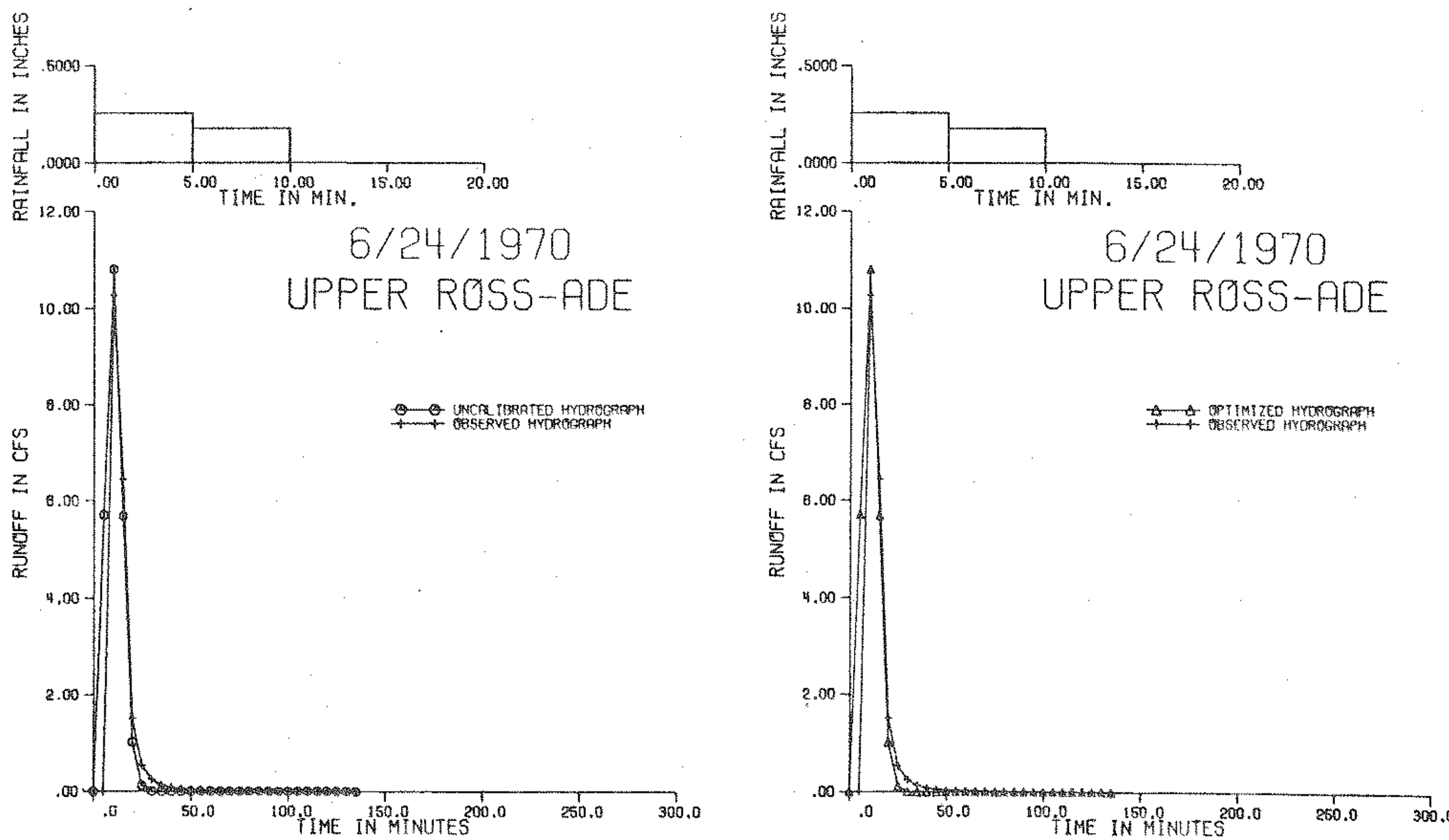


Figure 3.14 The Predicted Hydrographs from ILLUDAS. (Upper Ross-Ade Watershed Data)

Table 3.17 Comparison of the Predicted Hydrograph of Storm 6/24/1970 by ILLUDAS (Upper Ross-Ade Watershed Data)

	Peak (cfs)	Time to Peak (min)	Volume (in)	R_0	R	ISE
Predicted by Using Uncalibrated Parameter Values	10.8	10	0.066	0.77	0.88	30.11
Predicted by Using Optimal Parameter Values	8.4	10	0.050	0.92	0.96	18.03
Observed Values	10.3	10	0.055	--	--	--

3.7.4.2 Oakdale Avenue Basin

In eight out of eleven events in the Oakdale Avenue Basin, optimal parameters remain the same and hence these values are considered the "most probable" optimal estimates of these parameters. Typical observed and predicted hydrographs in which the arithmatic average and "most probable" parameter estimates used are shown in Fig. 3.15. The results of comparison between observed and calculated hydrographs are given in Table 3.18. The results obtained by using the "most probable" and the arithmatic average optimal parameter estimates are very close to each other.

3.7.5 Prediction Study Results from SWMM

3.7.5.1 Upper Ross-Ade Watershed

The average parameter estimates which are the same as the "most probable" values for this watershed were used to test the prediction performance of the model. The observed, and uncalibrated hydrographs, and hydrographs predicted by using optimal parameter estimates are plotted in Fig. 3.16. The results from the prediction study are summarized in Table 3.19. These results show that the hydrographs with optimal parameter estimates predict the flows better than the uncalibrated hydrographs.

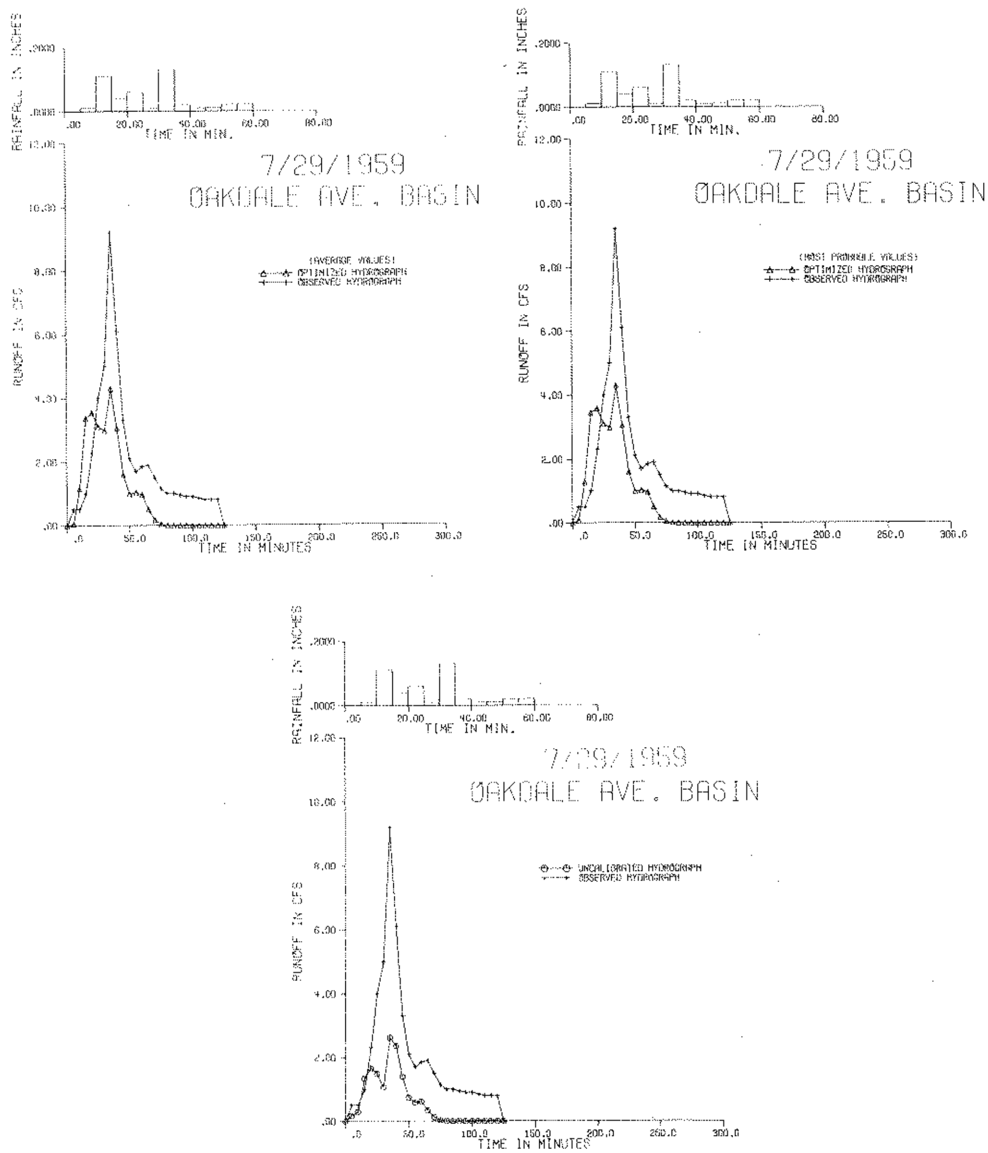


Figure 3.15 The Predicted Hydrographs from ILLUDAS. (Oakdale Avenue Basin Data)

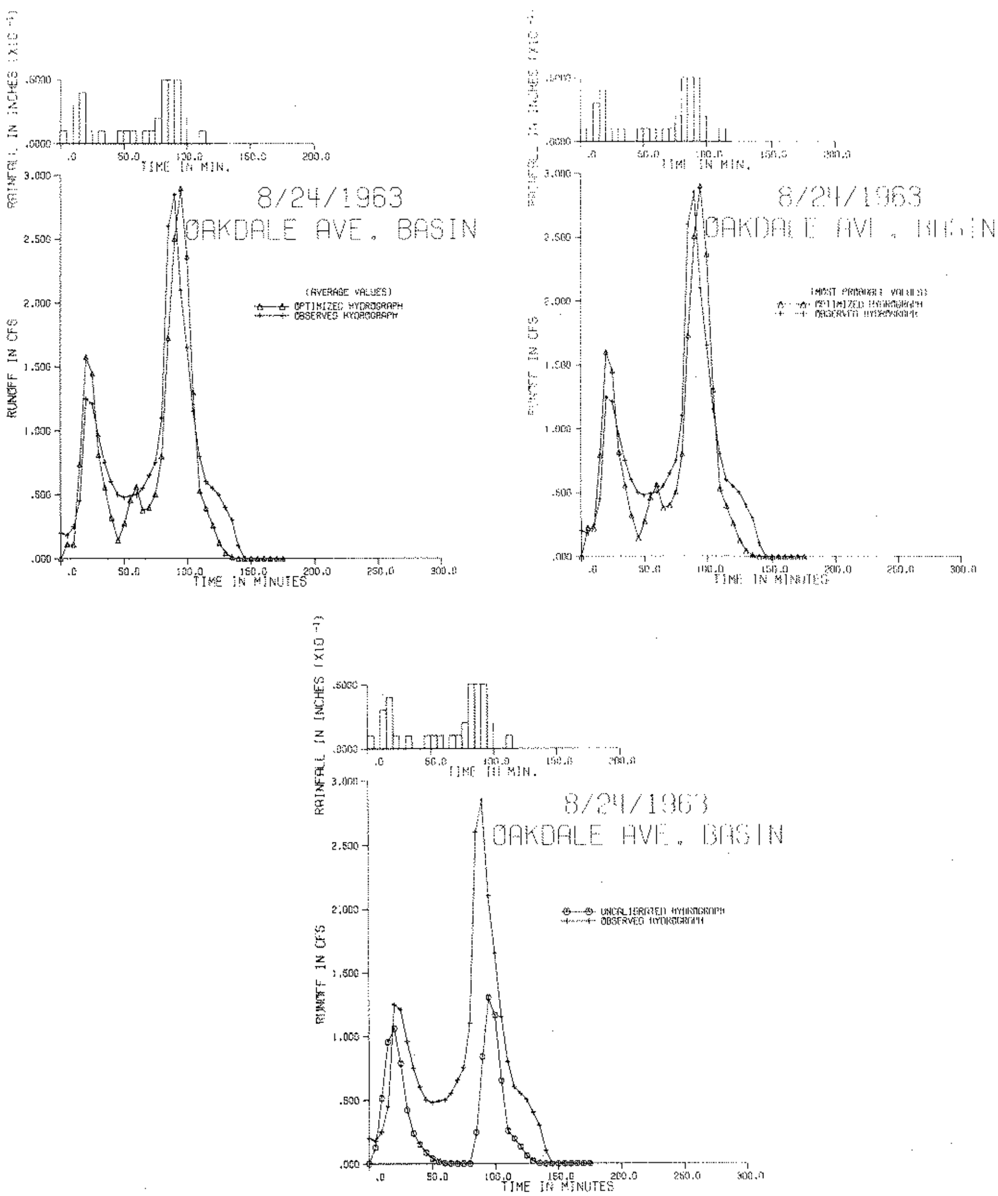


Figure 3.15, cont.

Table 3.18 Predicted Hydrographs by ILLUDAS. (Oakdale Avenue Basin Data)

		Peak (cfs)	Time to Peak (min)	Volume (in)	R_0	R	ISE
7/29/59	Uncalibrated	2.6	35	0.095	0.51	0.87	20.02
	Most Probable Optimized	4.3	35	0.175	0.69	0.78	15.97
	Average Optimized	4.3	35	0.173	0.69	0.79	15.90
	Observed	9.2	35	0.320	–	–	–
8/24/63	Uncalibrated	1.3	95	0.059	0.50	0.58	16.71
	Most Probable Optimized	2.9	95	0.139	0.88	0.90	8.02
	Average optimized	2.9	95	0.137	0.89	0.90	7.93
	Observed	2.8	90	0.157	–	–	–

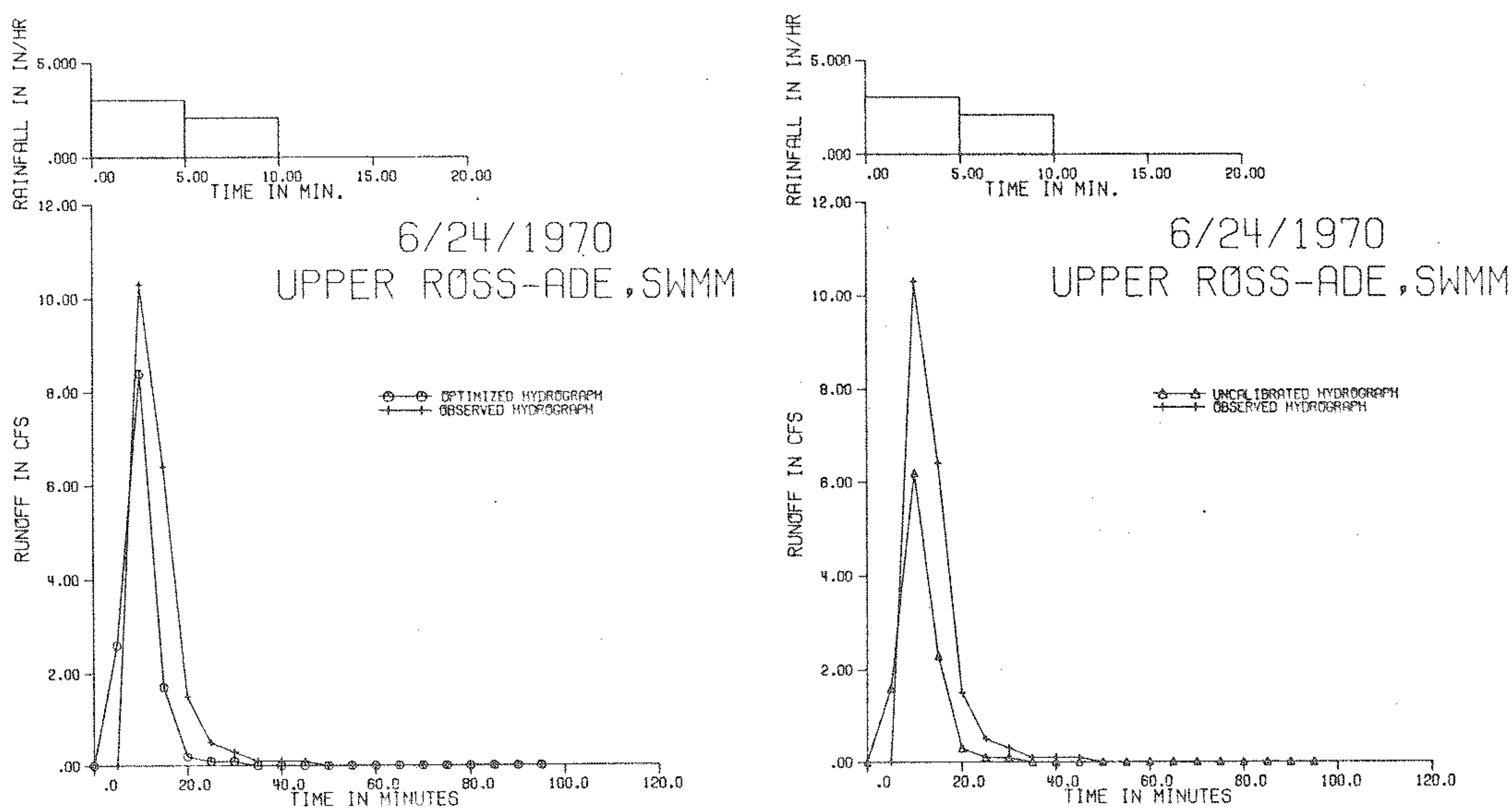


Figure 3.16 The Predicted Hydrographs from SWMM. (Upper Ross-Ade Watershed Data)

Table 3.19 Comparison of the Predicted Hydrographs from SWMM. (Upper Ross-Ade Watershed Data)

	Peak (cfs)	Time to Peak (min)	Volume (in)	R_0	R	ISE
Predicted on Uncalibrated Parameter Values	6.2	10	0.030	0.77	0.88	30.38
Predicted on Optimized Parameter Values	8.4	10	0.037	0.75	0.94	31.88
Observed Values	10.3	10	0.055	–	–	–

Table 3.20 Comparison of the Predicted Hydrographs from SWMM. (Oakdale Avenue Basin Data)

		Peak (cfs)	Time to Peak (min)	Volume (in)	R_0	R	ISE
7/29/59	Uncalibrated	9.7	35	0.273	0.72	0.80	17.35
	Optimized	9.7	35	0.305	0.85	0.90	12.66
	Observed	9.2	35	0.322	–	–	–
8/24/63	Uncalibrated	2.8	95	0.096	0.71	0.78	12.66
	Optimized	2.8	95	0.115	0.73	0.81	12.31
	Observed	2.9	100	0.157	–	–	–

3.7.5.2 Oakdale Avenue Basin

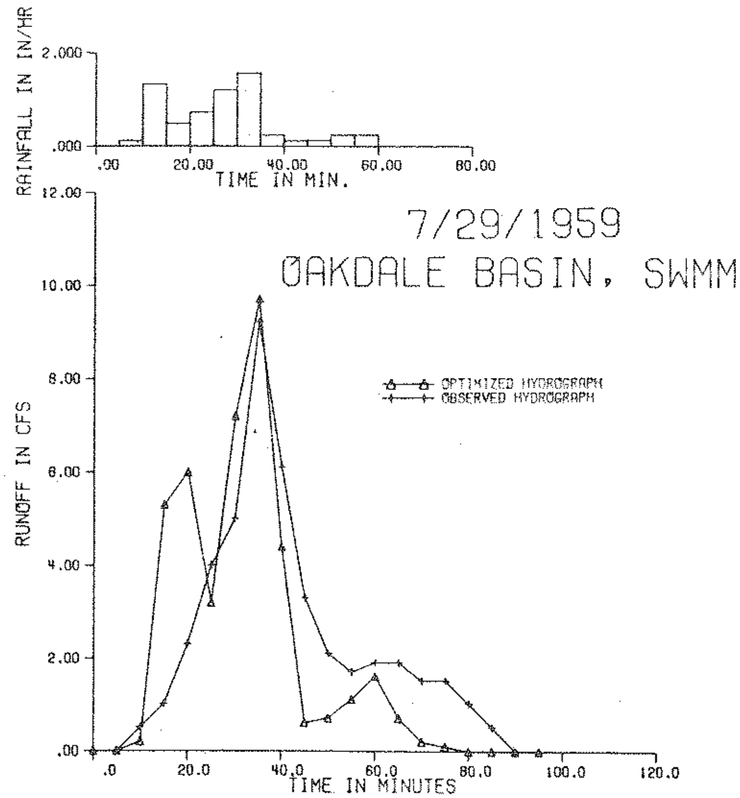
The observed, uncalibrated, and optimized hydrographs computed by using the arithmatic average of parameter estimates which are identical to the "most probable" values in this watershed are shown in Fig. 3.17. The results from the comparison of the observed, uncalibrated, and optimized hydrographs are given in Table 3.20. Also listed in Table 3.20 are the statistics of agreement between the performance of the predicted and observed hydrographs. The prediction performance of the SWMM model is not as good as that of the ILLUDAS model.

3.7.6 Prediction Study Results from MINNOUR

The prediction performance of the MINNOUR model was tested by using the average values generated from the regeneration study. By using these parameter values, the predicted hydrographs are plotted against the observed hydrographs. Typical results are shown as Fig. 3.18. The peak flows, times to peak, and volumes are given in Table 3.21. The results from the prediction study, although acceptable, are not quite as good as those from the regeneration study.

3.8 Summary

Three optimal hydrologic models, ILLUDAS, Runoff Block of SWMM, and MINNOUR were investigated. Each of these models



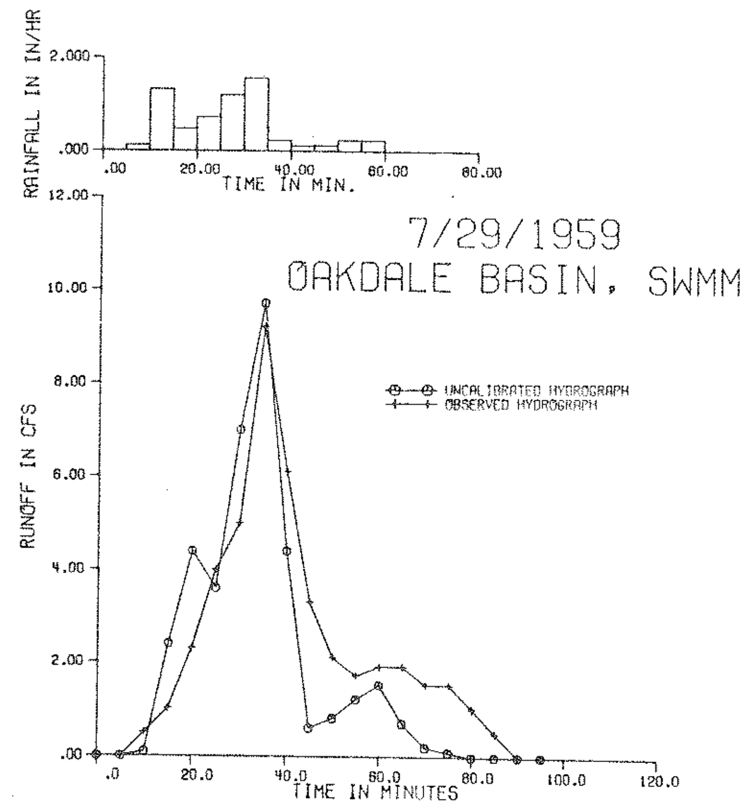


Figure 3.17 The Predicted Hydrographs from SWMM. (Oakdale Avenue Basin Data)

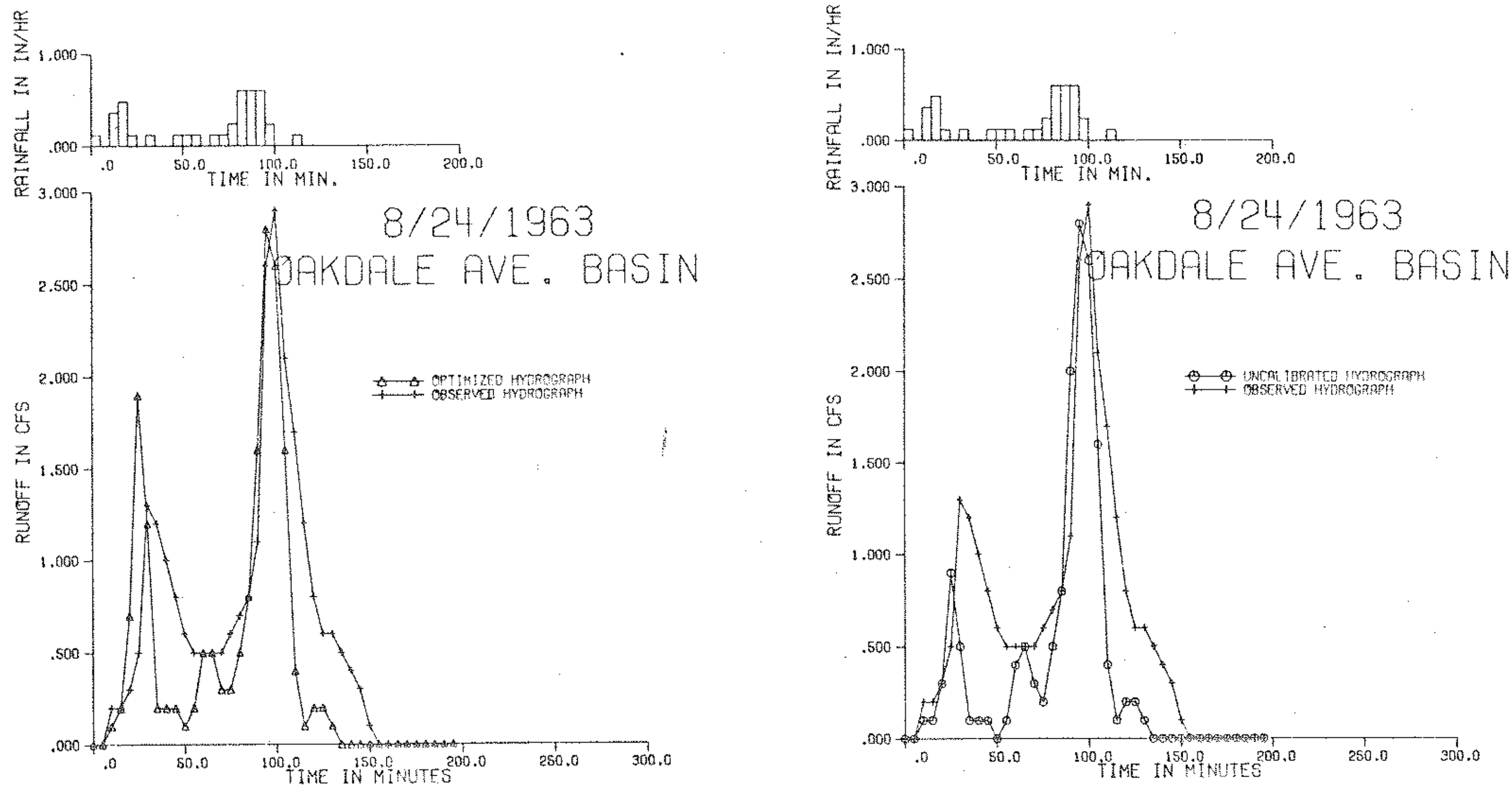


Figure 3.17, cont.

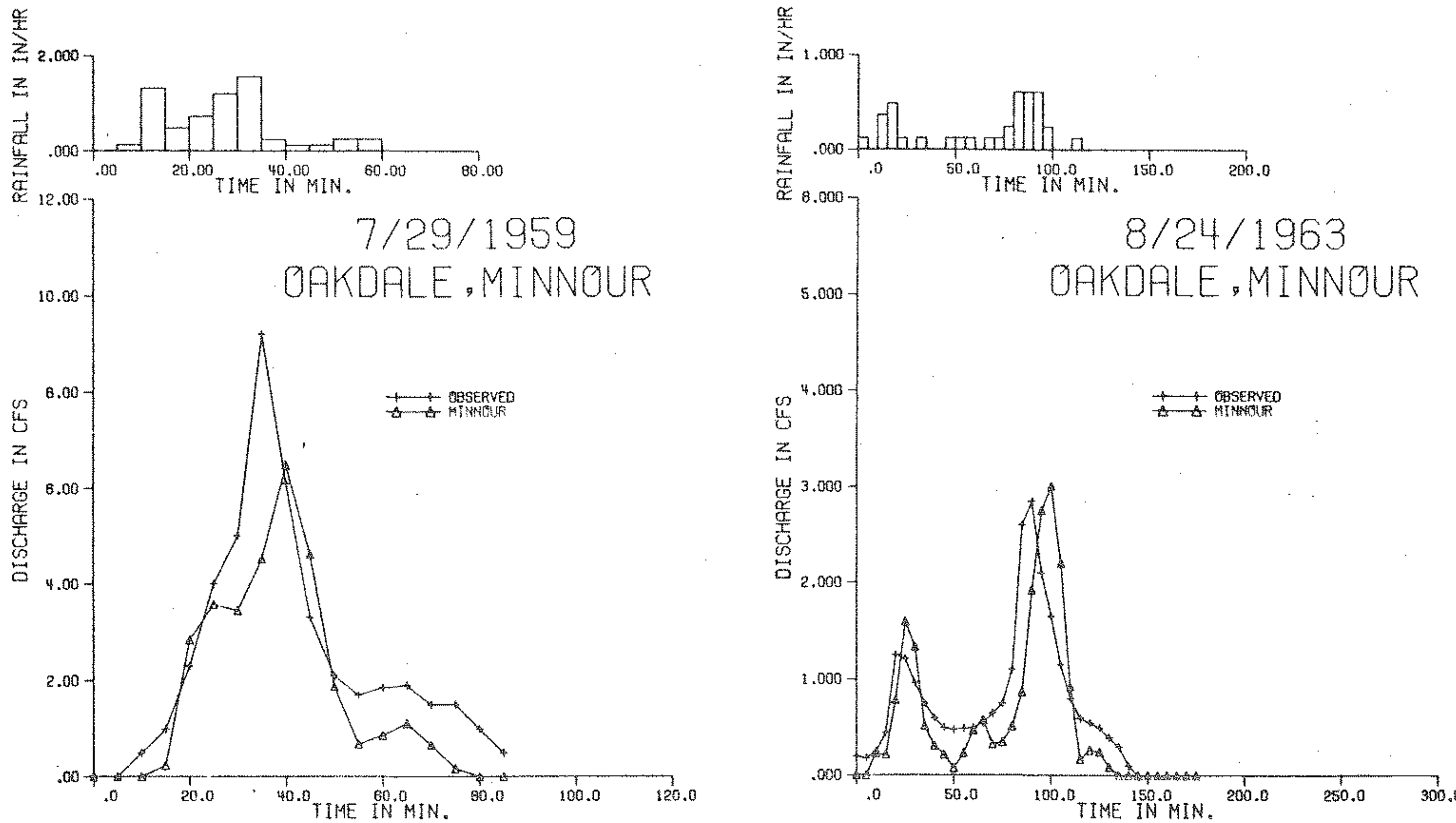


Figure 3.18 The Predicted Hydrographs from MINNOUR.
(Oakdale Avenue Basin Data)

Table 3.21 Predicted Hydrographs by MINNOUR

		Peak (cfs)	Time to Peak (min)	Volume (in)
7/29/1959	MINNOUR Observed	6.48 9.20	40 35	0.199 0.322
8/24/1963	MINNOUR Observed	3.01 2.85	100 90	0.130 0.157

has its own merits. MINNOUR model is the simplest if the model structure is considered. It is based on the Soil Conservation Service Triangular Unit Hydrograph method and the Holtan infiltration model. The optimization technique used with MINNOUR is a modification of the Box-Complex method. The optimization technique can assist the user in determining the "best" set of input parameters. With the appropriate hydrologic input parmeters, the MINNOUR model gave consistently good results in the regeneration study. The results of the prediction study, although acceptable, are not as good as those from the regeneration study.

The ILLUDAS model may be used to design or to evaluate existing storm sewer systems. The detention storage requirement may also be estimated by using it. The input data requirements of ILLUDAS are greater than that of MINNOUR but many of the inputs can be easily determined from readily available information. In the present study, the modified Rosenbrock's optimization technique is used to estimate the parameters of the ILLUDAS model. The ILLUDAS model has been extensively investigated by many and its performance is, in general, good. It has been gaining popularity among the designers and planners. Incorporating the optimization calibration scheme makes the model more versatile and easier to apply. The model is tested and good results are obtained in both the regeneration and prediction studies.

The model structure of the Runoff Block of SWMM is fairly complex. The model may be used to simulate the quantity and quality of runoff. Only storm water quantity is estimated in the present study. The data requirements for SWMM are extensive. This model has been improved in the present study by adding Rosenbrock's optimization method to estimate some of its parameters. The results obtained from the model may be considered to be fair.

These models are applied to two watersheds, the Upper Ross-Ade Watershed and the Oakdale Avenue Basin. Table 3.22 show the results of the regeneration study. If only the regeneration study is considered, all three models performed well. Although the MINNOUR model is the simplest as far as its structure is concerned, it performed best in regenerating the peak flows and volumes. The ILLUDAS model and the SWMM model also gave good results. Although some variation may be expected from the results reported here because of the differences in watershed conditions and storms, the results presented in the present study are typical.

For the purpose of prediction, the average values of the parameters calibrated are used as the final parameter value and the models were tested by using other storms which were not used in the regeneration study. The results of the prediction study are summarized in Table 3.23. The results shown in Table 3.23 do not clearly indicate the superiority

Table 3.22 Comparison of the Regenerated Hydrographs from Three Models

	Storm Date M/D/Y	Observed		MINNOUR		ILLUDAS		SWMM	
		Peak	Volume	Peak	Volume	Peak	Volume	Peak	Volume
Upper Ross-Ade Watershed	7/18/70	15.9	0.33	N/A	N/A	24.4	0.41	23.0	0.62
	7/30/70	8.4	0.41	N/A	N/A	7.0	0.41	6.1	0.61
	9/04/70	12.3	0.23	N/A	N/A	13.0	0.23	14.9	0.36
Oakdale Avenue Basin	5/19/59	6.5	0.26	7.4	0.23	7.5	0.26	7.1	0.25
	7/02/60	4.6	0.14	4.4	0.12	4.7	0.14	4.3	0.13
	7/26/60	4.3	0.42	2.6	0.31	2.5	0.31	2.4	0.28
	9/18/60	5.1	0.39	5.1	0.27	5.2	0.28	4.3	0.25
	10/14/60	4.5	0.12	4.6	0.08	5.2	0.10	4.6	0.09
	4/17/63	6.5	0.35	6.8	0.28	6.1	0.29	5.5	0.27
	4/19/63	11.6	0.22	10.4	0.18	9.1	0.22	9.2	0.18
	4/29/63	6.7	0.75	6.6	0.57	5.0	0.54	4.5	0.48
	6/19/63	7.8	0.24	7.6	0.14	5.4	0.12	4.9	0.10
	8/02/63	5.9	0.25	5.2	0.23	5.9	0.25	5.6	0.23
	9/22/64	4.4	0.18	3.9	0.14	4.7	0.17	4.1	0.15
Statistical Measurement	R_0	–	–	0.99	0.94	0.96	0.93	0.95	0.89
	R	–	–	0.95	0.98	0.81	0.96	0.85	0.94
	ISE	–	–	3.63	8.54	6.57	8.87	7.35	11.35

Table 3.23 Comparison of the Predicted Hydrographs from Three Models

	Storm Date M/D/Y	Observed		MINNOUR		ILLUDAS		SWMM	
		Peak	Volume	Peak	Volume	Peak	Volume	Peak	Volume
Upper Ross-Ade Watershed	6/24/70	10.3	0.055	N/A	N/A	8.4	0.050	8.4	0.037
Oakdale Avenue Basin	7/29/59 8/24/63	9.20 2.9	0.32 0.16	6.5 3.0	0.20 0.13	4.3 2.9	0.17 0.14	9.7 2.8	0.31 0.12

of one model over others. The complexity of model structure does not guarantee better performance. In the present study, SWMM gave fair results and MINNOUR and ILLUDAS gave good results.

CHAPTER IV

ANALYSIS OF OBJECTIVE FUNCTIONS USED IN OPTIMAL PARAMETER ESTIMATION OF URBAN RUNOFF MODELS

4.1 Introduction

An index of agreement between the observed and calculated hydrographs is needed in order to estimate the parameters of urban rainfall-runoff models. This index is usually called the objective function. The selection of objective functions, to a certain extent, depends on the problem. For example, for the design of detention storage facilities, the objective function must emphasize the total runoff volume under the hydrograph. For the problem of sizing drainage system pipes, it is better to define an objective function which would emphasize the peak flows and shape of design hydrographs. In the overall investigation of a drainage system, an objective function which has two components, emphasizing both the shape and the volume of the hydrograph, may be suitable.

The selection of an objective function for the optimal parameter estimation of urban rainfall-runoff models is, to a certain extent, a subjective process. The choice of the objective function, however, affects the optimal parameter estimates of the model. The optimal parameter estimates are obviously optimal only with respect to the objective function used for optimization.

Very few investigators have examined the problem of selection of objective functions which are used with the rainfall-runoff models. Those who have investigated the problem, as discussed below, have often used only simple rainfall-runoff models. Consequently, the basic objective of the research discussed in this chapter is to investigate several objective functions using an urban rainfall-runoff model and to arrive at some conclusions about these objective functions.

4.2 Literature Review

Problems related to selecting objective functions to be used with hydrologic models have been discussed by some earlier investigators although not in great detail. One of the earliest discussions of this problem was by Dawdy and O'Donnell (1965). They used the sum of the squared deviations between the observed and calculated runoff in their work with a catchment model. They also suggested other objective functions and a combination of objective functions which may be used with hydrologic models. Dawdy and Lichty (1968) used the criterion of the minimization of the sum of the squares of the differences between the logarithms of estimated and logarithms of the measured discharges to estimate the model parameters. Both of these measures are dimensional and depend on the number of observations. Ibbitt and O'Donnell (1971) recommended a normalized objective function NU:

$$NU = \frac{(U/n)^{0.5}}{(Q/n)} = \frac{(nU)^{0.5}}{Q} \tag{4.1}$$

where U is the basic objective function:

$$U = \sum_{i=1}^{n} (Qobs_i - Qcal_i)^2 \tag{4.2}$$

and n is the number of the hydrograph ordinates, Q is the sum of the observed hydrograph ordinates, and $Qobs_i$ and $Qcal_i$ are respectively the observed and calculated hydrograph ordinates. Nash and Sutcliffe (1970) proposed another measurement criterion, efficiency R^2, which is defined as :

$$R^2 = \frac{F^2{}_0 - F^2}{F^2{}_0} \tag{4.3}$$

where $$F^2_0=\sum_{i=1}^{n} (Qobs_i-QBAR)^2 \quad (4.4)$$

and QBAR is the mean of the observed hydrograph ordinates and

$$F^2=\sum_{i=1}^{n} (Qcal_i-Qobs_i)^2 \quad (4.5)$$

This efficiency parameter R^2 was also used by Bosman et al. (1975) along with another objective function which was the sum of squares of logarithms of deviations between observed and estimated flows. The logarithms of the deviations rather than the flows were used to prevent the parameters from being biased to fit only large magnitude events. Lichty et al. (1968) adopted this concept and used an objective function U_3 which included both peak and volume component of the hydrograph. The objective function U_3 was weighted to emphasize the peak and has a form as follows:

$$U_3=U_1+0.5U_2 \quad (4.6)$$

where $$U_1=\sum_{j=1}^{m}(\ln(PC_j)-\ln(PO_j))^2 \tag{4.7}$$

$$U_2=\sum_{j=1}^{m}(\ln(VC_j)-\ln(VO_j))^2 \tag{4.8}$$

Carrigan (1973) extended the model of Lichty et al. (1968) and developed a calibration program which has the options to minimize U_1, U_2, and U_3 which are defined above and U_4 which is defined in Eq. (4.9)

$$U_4=\sum_{j=1}^{m}(\ln(PC_j-\frac{VO_j}{VC_j})-\ln(PO_j))^2 \tag{4.9}$$

in which PC_j: calculated peak flows

PO_j: observed peak flows

VC_j: calculated volumes

VO_j: observed volumes

m : number of storms

More recently, Jewell et al. (1978) calibrated the SWMM model using the criterion function based on the observed and calculated volumes and on the differences between the sum of the observed and calculated peak flows for all the storm events investigated. Another criterion used by them is the so called standard error of estimation (SEE):

$$SEE=\left(\frac{\sum_{i=1}^{n} (Qcal_i - Qobs_i)^2}{n-2}\right)^{0.5} \tag{4.10}$$

Although a model may be calibrated by using a trial and error method, an objective function is still needed for optimal parameter estimation. Sugawara (1979) calibrated the tank model by using a computer program and trial and error methods. The objective functions used by Sugawara are given below.

$$RQ(I)=\sum_{N} Qcal(N) / \sum_{N} Qobs(N) \tag{4.11}$$

$$RD(I)=-\frac{\sum'_{N} [\log Qcal(N-1) - \log Qcal(N)]}{\sum'_{N} [\log Qobs(N-1) - \log Qobs(N)]} \tag{4.12}$$

where Qobs is the observed discharge, Qcal is the calculated discharge, I is the index number of the subperiod, N is the day number, Σ is the sum over the days belonging to the subperiod I and Σ′ is the sum over the days belonging to the subperiod I for which Qcal(N-1)-Qcal(N) is positive. The parameters of the tank model were estimated by using the criteria in Eqs. (4.11) and (4.12).

Wildermuth and Yeh (1979) used an objective function which has two components as indicated below.

$$U=\alpha\sum_{i=1}^{n}(\log(Qobs_i)-\log(Qcal_i))^2+\beta(\log(VO)-\log(VC))^2 \qquad (4.13)$$

The first of these components is the sum of the squares of the deviations between logarithms of the observed and calculated discharges, the second component is the calculated logarithms of the runoff volume.

Diskin et al. (1978) used the relative mean absolute deviation to calibrate their parallel cascades model. The relative mean absolute deviation is defined as in Eq. (4.14)

$$U=\frac{(1/n)\sum_{i=1}^{n}|(Qcal_i-Qobs_i)|}{Q_p} \qquad (4.14)$$

in which Q_p is the peak flow of runoff hydrograph. In the work mentioned above, different forms of objective functions are used. However, the reasons for selection of particular forms of objective function are seldom discussed. Wood (1975), for example, examined the general objective function shown in Eq. (4.15)

$$U = \sum_{i=1}^{n} \frac{\left(Qobs_i - Qcal_i\right)^k}{\left(Qobs_i\right)^n} \tag{4.15}$$

and concluded that splitting the parameters into two groups and performing sequential optimization with different objective functions (n=0, k=1., n=2, k=2., and n=0, k=2) is a useful method.

Johnston and Pilgrim (1976) investigated the general form of the objective function (Eq. (4.16)) used in many parameter estimation methods

$$U = \sum_{i=1}^{n} \left|Qobs_i - Qcal_i\right|^j \tag{4.16}$$

They discussed two types of U which are used in forming the objective function. These are (1) absolute values of deviations between observed and calculated runoff are raised to powers j before summation, and (2) the observed and calculated runoff are transformed in some way before the calculation of the deviations which are then raised to power 2 and summed. Values of ½, 1, and 2 for j were tested in the single reservoir model. They concluded that the sum of the squares of the deviations was the best objective function. When transformed runoff values are used before calculating the deviation, the parameter estimates depended on the transformation used.

Perhaps the most comprehensive analysis of the procedure for selection of the objective functions was carried out by Diskin and Simon (1977). They investigated the effects of selecting different objective functions and developed guidelines and recommendations for the selection procedure so that the subjectivity involved is reduced. They concluded that better results are obtained if the objective function is chosen according to the engineering application for which the results will be used. Their procedure was tested by using a rather simple hydrologic model. Loganathan (1979) conducted a limited study along the lines of Diskin and Simon (1977). He tested the method on a runoff simulation model (Kite (1978)). The procedure adopted in the present study is very similar to that of Diskin and Simon (1977). The methodology used in the present study is discussed in the next section. The application of the method and results are discussed in section 4.4.

4.3 Methodology

The set of parameter estimates P_j obtained from a model and a data set is optimal only for the objective function U_j used for parameter estimation. The value of another objective function U_k ($k \neq j$) computed by using the set of parameter values P_j would be different from U_j. A matrix may be formed using the values obtained from various objective functions. For example, the elements of the first column of such a matrix consist of the values calculated for all the objective functions U_j using the values P_1 which are derived by using objective function U_1. The second column would consist of objective function values computed by using P_2 and so on. It is clear that the diagonal elements of this matrix should have the smallest values in each row as they represent the optimal values estimated by using that particular objective function. The difference between the elements in any row shows the degree of the closeness of the parameter estimates. Fig. 4.1 shows the set up of the matrix. The matrix may be further transformed. Instead of dealing with the actual objective function values calculated, the elements may be ranked based on their relative magnitudes along each row. The lowest value is assigned 1, the next higher value 2, and the rank is increased by unity as the magnitude of the element increases. The diagonal elements of the rank matrix obviously have a value of 1.

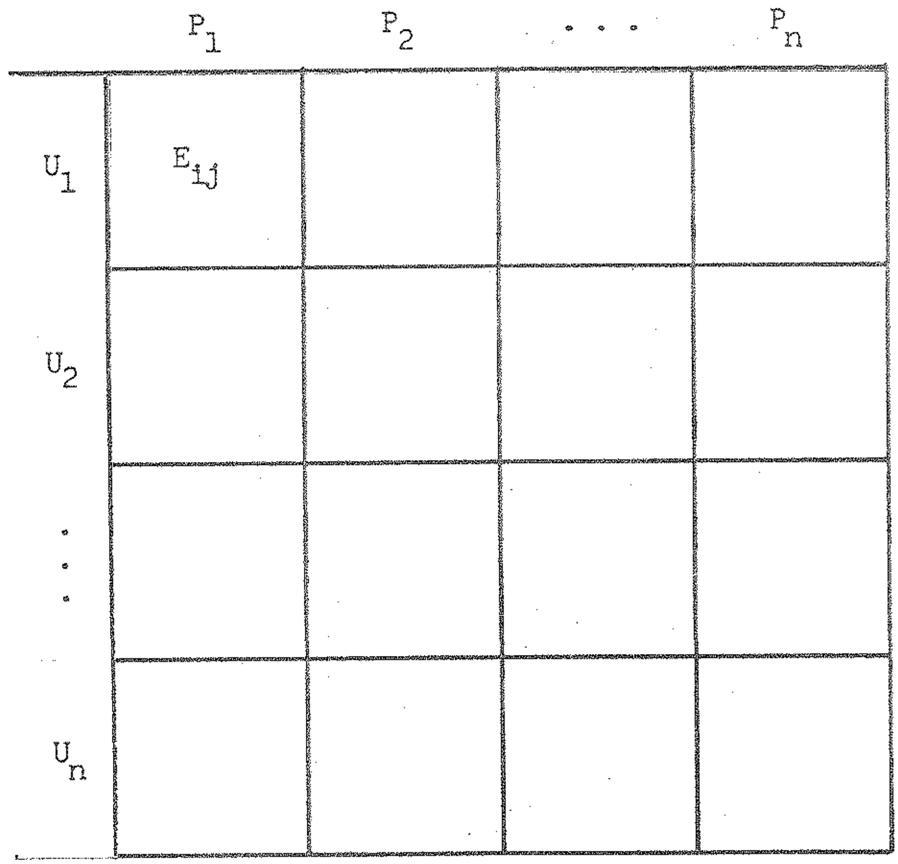


Figure 4.1 The Matrix for E_{ij}

Instead of ranking the objective function values they may be normalized along rows and these normalized values may be used instead of ranks. The normalized value is the actual value minus the mean value and divided by the standard deviation of objective function values along a row. The closeness of objective function values may be easily seen by comparing normalized values.

A set of optimal values P_j which produces the lowest column sum of the normalized values is considered as the best among the set of objective functions examined. The procedure clearly depends on the data and perhaps the model used in the study. If different data measured on different watersheds are analyzed by using the procedure discussed above, a merit number which indicates the performance of the objective function for a specific storm event and watershed would be assigned in an ascending order starting with 1 for the objective function chosen as the best. The sum of the merit number over the storm events and watersheds for the objective function are used to select the "best" objective functions. The least sum indicates the most favorable among all the storm events and watershed investigated.

This procedure was used with ILLUDAS and data from Upper Ross-Ade and Oakdale Avenue Watersheds and the results are discussed below.

4.4 Discussion of Results

The methodology discussed above was used with ILLUDAS. Part of the data described in Chapter II was used in the analysis. The optimization scheme discussed in Chapter III was used to compute optimal parameter estimates.

Two sets of objective functions were investigated. The first set, shown in Table 4.1, was the same as that investigated by Diskin and Simon (1977). This set was investigated mainly to determine how the results of the present study in which ILLUDAS is used compares with the results obtained by Diskin and Simon. The set of objective functions in Table 4.1 does not include some of the objective functions which have been used previously in hydrologic modeling. It was decided to consider them also. The results from these two sets of objective functions are also compared.

4.4.1. Results from the First Set of Objective Functions

The optimal parameter values derived for each objective function and the corresponding objective function values are tabulated as Table 4.2 for the storm of April 29, 1963. The same initial, upper, and lower values listed in Chapter III for each of the parameters were used in parameter estimation. Some of the objective functions did not converge to a value at first trial. For these cases, the process was repeated by using the values obtained from the

Table 4.1 The First Set of Twelve Objective Functions Used

$$U_1 = \Sigma(Y_i - X_i)^2$$

$$U_2 = \Sigma[\{2(Y_i - X_i)\}/(Y_i + X_i)]^2$$

$$U_3 = [\{N\Sigma(Y_i - X_i)^2\}^{1/2}]/\Sigma Y_i$$

$$U_4 = (\Sigma|Y_i - X_i|)/\Sigma Y_i$$

$$U_5 = |\Sigma(Y_i - X_i)|/\Sigma Y_i$$

$$U_6 = (1/N)\Sigma[\{2(Y_i - X_i)\}/(Y_i + X_i)]^2$$

$$U_7 = [\Sigma(Y_i - X_i)^2]/[\Sigma(Y_i - \overline{Y})^2]$$

$$U_8 = \left[\{\Sigma(Y_i{}^{1/3} - X_i{}^{1/3})^2\}^{3/2}\right]/(N^{1/2}\Sigma Y_i)$$

$$U_9 = 2|(\overline{Y} - \overline{X})/(S_y + S_x)| + 2|(V_y - V_x)/(V_y + V_x)| + 2|(\overline{Y} - \overline{X})/(\overline{Y} + \overline{X})|$$

$$U_{10} = U_9 + |C_{sy} - C_{sx}|$$

$$U_{11} = U_{10} + |C_{ky} - C_{kx}|$$

$$U_{12} = \left[\Sigma(Y_i{}^{1/2} - X_i{}^{1/2})^2\right]/\Sigma Y_i$$

where:

Y_i := the observed values

X_i := the calculated values

$\overline{X}, \overline{Y}$ = the mean of the series X_i, Y_i, respectively

V_x, V_y = the variance

S_x, S_y = the standard deviation

C_{sx}, C_{sy} = the coefficient of skew

C_{kx}, C_{ky} = the coefficient of kurtosis

Table 4.2 Final Optimal Parameter Values for Storm 4/29/1963

Objective Function	Optimal Parameter Value						Final Objective Function Values
	ABSTRT	DEPG	FI	FO	FC	KIF	
1	0.067	0.050	0	7.933	0.279	2.407	29.85
2	0.200	0.197	0	7.949	0.399	1.832	47.41
3	0.067	0.050	0	7.933	0.279	2.407	0.3991
4	0.067	0.050	0	7.933	0.279	2.407	0.3311
5	0.000	0.050	0	7.933	0.279	2.407	0.2900
6	0.200	0.197	0	7.949	0.399	1.832	0.6495
7	0.067	0.050	0	7.933	0.279	2.407	0.2125
8	0.133	0.050	0	7.933	0.279	2.407	0.0096
9	0.000	0.050	0	7.933	0.279	2.407	1.0293
10	0.067	0.135	0	7.813	0.461	2.335	1.3023
11	0.067	0.135	0	7.813	0.461	2.335	2.1265
12	0.067	0.050	0	7.933	0.279	2.407	0.0677

first trial and repeated until the convergence criterion was met. Convergence problems were present with objective functions 10 and 11.

Using the set of parameter values listed in Table 4.2 for various objective functions, a matrix with elements E_{ij} is calculated by using the optimal parameter estimates for every objective function. This matrix is shown in Table 4.3. The diagonal elements E_{ii} of the matrix are the smallest elements in each row. If this were not the case, then only a local minimum has been found for the objective function i rather than the global optimum value. If a local minimum has been identified, the global minimum value of objective function i can be estimated by continuing the search. In the case presented in Table 4.3, this difficulty was not encountered. In Table 4.3, there are several identical columns which result from identical optimal parameter values. If two objective functions produce identical results they are redundant. If more than one objective function gives identical values, the simplest objective function among them will be used. In the case presented in Table 4.3, U_1 may be used to represent U_1, U_3, U_4, U_7, U_{10}, U_{11}, and U_{12}; U_2 to represent U_2 and U_6; and U_5 to represent U_5 and U_9. The number of objective functions considered may thus be reduced from twelve to four. Only the four objective functions given below in Eqs. (4.17) to (4.20) were analyzed further.

Table 4.3 Matrix E_{ij} of the Storm of 4/29/1963

i \ j	1	2	3	4	5	6	7	8	9	10	11	12
1	29.85	67.93	29.85	29.85	32.75	67.93	29.85	52.70	32.75	29.85	29.85	29.85
2	51.97	47.41	51.97	51.97	53.22	47.41	51.97	47.44	53.22	51.97	51.97	51.97
3	0.2991	0.6021	0.3991	0.3991	0.4181	0.6021	0.3991	0.5303	0.4181	0.3991	0.3991	0.3991
4	0.3311	0.4144	0.3311	0.3311	0.3381	0.4144	0.3311	0.3841	0.3381	0.3311	0.3311	0.3311
5	0.3255	0.3934	0.3255	0.3255	0.2900	0.3934	0.3255	0.3580	0.2900	0.3255	0.3255	0.3255
6	0.7120	0.6495	0.7120	0.7120	0.7290	0.6495	0.7120	0.6498	0.7290	0.7120	0.7120	0.7120
7	0.2125	0.4837	0.2125	0.2125	0.2332	0.4837	0.2125	0.3725	0.2125	0.2125	0.2125	0.2125
8	0.0117	0.0105	0.0117	0.0117	0.0127	0.0105	0.0117	0.0096	0.0127	0.0117	0.0117	0.0117
9	1.1632	1.7855	1.1632	1.1632	1.0293	1.7855	1.1632	1.4924	1.0293	1.1632	1.1632	1.1632
10	1.3023	2.3392	1.3023	1.3023	1.3350	2.3392	1.3203	2.0258	1.3350	1.3023	1.3023	1.3023
11	2.1265	4.7191	2.1265	2.1265	2.6284	4.7191	2.1265	4.4390	2.6284	2.1265	2.1265	2.1265
12	0.0677	0.0875	0.0677	0.0677	0.0731	0.0875	0.0677	0.0769	0.0731	0.0677	0.0677	0.0677

$$U_1=\Sigma(Y_i-X_i)^2 \tag{4.17}$$

$$U_2=\Sigma[(2(Y_i-X_i))/(Y_i+X_i)]^2 \tag{4.18}$$

$$U_5=|\Sigma(Y_i-X_i)|/\Sigma Y_i \tag{4.19}$$

$$U_8=[(\Sigma(Y_i^{0.333}-X_i^{0.333})^2)^{1.5}]/(N^{0.5}\Sigma Y_i) \tag{4.20}$$

The E_{ij} values for the four objective functions and their respective ranks R_{ij} and the normalized values along the row elements are shown in Table 4.4 for the storm of April 29, 1963 on Oakdale Avenue Basin. Also indicated in Table 4.4 are the sums of the normalized values for each of the columns. The smallest column sum indicates the "best" objective function. For example in Table 4.4 the optimal parameter values derived from objective function 8 gives the best overall performance among the objective functions for this particular storm and model investigated.

The objective function value estimates for five other storms on Oakdale Avenue Basin and for 3 storms on Upper Ross-Ade Watershed are summarized in Tables 4.5 through 4.12. As expected, the ranks for objective functions vary with the storm event. For some storms identical column values result. In such cases, same ranks are assigned to the objective functions.

Table 4.4 Objective Function, Ranks, and Normalized Values of Storm of 4/29/1963

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	29.85	67.93	32.75	52.70
	R	1	4	2	3
	N	-0.89	1.23	-0.73	0.38
	2	51.97	47.41	53.22	47.44
	R	3	1	4	2
	N	0.65	-0.86	1.06	-0.85
	5	0.3255	0.3934	0.2900	0.3580
	R	2	4	1	3
	N	-0.0422	0.1344	-0.1345	0.0423
	8	0.0117	0.0105	0.0127	0.0096
	R	3	2	4	1
	N	0.0459	-0.0500	0.1259	-0.1219
	Total R	9	11	11	9
	N	-0.2363	0.4544	0.3214	-0.5496
	Final Ranks	2	4	3	1

N: Normalized Values

R: Ranks

Table 4.5 Objective Function, Ranks, and Normalized Values of Storm of 5/19/1959

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	12.94	35.30	51.41	18.38
	R	1	3	4	2
	N	-0.4339	0.1699	0.6049	-0.3410
	2	19.11	18.68	23.15	19.66
	R	2	1	4	3
	N	-0.0459	-0.0650	0.1326	-0.0217
	5	0.0161	0.0383	0.0001	0.2566
	R	2	3	1	4
	N	-0.4156	-0.2660	-0.5235	1.2051
	8	0.0064	0.0119	0.0198	0.0058
	R	2	3	4	1
	N	-0.3296	0.0666	0.6357	-0.3728
	Total R	7	10	13	10
	N	-1.2250	-0.0945	0.8497	0.4696
	Final Ranks	1	2	4	3

N: Normalized Values

R: Ranks

Table 4.6 Objective Function, Ranks, and Normalized Values of Storm of 7/02/1960

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	5.310	5.830	15.51	5.580
	R	1	3	4	2
	N	-0.2630	-0.2318	0.7315	-0.2367
	2	10.42	10.23	12.18	10.24
	R	3	1	4	2
	N	-0.0284	-0.0441	0.1158	-0.0433
	5	0.2150	0.2570	0.0011	0.2540
	R	2	4	1	3
	N	0.1402	0.3174	-0.7623	0.3047
	8	0.0034	0.0033	0.0074	0.0032
	R	3	2	4	1
	N	-0.1761	-0.1951	0.5854	-0.2142
	Total R	9	10	13	8
	N	-0.3273	-0.1536	0.6704	-0.1895
	Final Ranks	1	3	4	2

N: Normalized Values

R: Ranks

Table 4.7 Objective Function, Ranks, and Normalized Values of Storm of 7/28/1960

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	20.07	20.15	25.36	20.08
	R	1	3	4	2
	N	-0.0558	-0.0525	0.1638	-0.0554
	2	64.83	64.79	67.11	64.87
	R	2	1	4	3
	N	-0.0078	-0.0083	0.0234	-0.0072
	5	0.2987	0.3035	0.2452	0.2968
	R	3	4	1	2
	N	0.0394	0.0544	-0.1273	0.0335
	8	0.0068	0.0069	0.0077	0.0068
	R	1	2	3	1
	N	-0.0271	-0.0248	0.0789	-0.0271
	Total R	7	10	12	8
	N	-0.0513	-0.0310	0.1388	-0.0562
	Final Ranks	2	3	4	1

N: Normalized Values

R: Ranks

Table 4.8 Objective Function, Ranks, and Normalized Values of Storm of 10/14/1960

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	18.29	18.85	32.57	18.85
	R	1	2	3	2
	N	-0.1497	-0.1280	0.4056	-0.1280
	2	17.02	16.02	19.62	16.02
	R	2	1	3	1
	N	-0.0078	-0.0597	0.1271	-0.0597
	5	0.5094	0.5744	0.1975	0.5744
	R	2	3	1	3
	N	0.0875	0.2061	-0.4997	0.2061
	8	0.0420	0.0400	0.0630	0.0400
	R	2	1	3	1
	N	-0.0803	-0.1181	0.3166	-0.1181
	Total R	7	7	10	7
	N	-0.1503	-0.0997	0.3496	-0.0997
	Final Ranks	1	2	3	2

N: Normalized Values

R: Ranks

Table 4.9 Objective Function, Ranks, and Normalized Values of Storm of 4/19/1963

	i \ j	1	2	5	8
Oakdale Avenue Basin	1	92.39	108.1	100.8	108.1
	R	1	3	2	3
	N	-0.0863	0.0497	-0.0142	0.0497
	2	58.17	16.08	57.27	16.08
	R	3	1	2	1
	N	0.4454	-0.4360	0.4266	-0.4360
	5	0.2912	0.1155	0.0024	0.1155
	R	3	2	1	2
	N	0.8674	-0.0828	-0.6812	-0.0828
	8	0.1314	0.0240	0.0803	0.0240
	R	3	1	2	1
	N	0.935	-0.4691	0.0031	-0.4691
	Total R	7	6	10	7
	N	2.1419	-0.9382	-0.2667	-0.9382
	Final Ranks	3	1	2	1

N: Normalized Values

R: Ranks

Table 4.10 Objective Function, Ranks, and Normalized Values of Storm of 7/15/1970

	i \ j	1	2	5	8
Upper Ross-Ade Watershed	1	492.7	492.7	537.8	537.8
	R	1	1	2	2
	N	-0.0390	-0.0390	0.0390	0.0390
	2	251.1	251.1	252.4	252.4
	R	1	1	2	2
	N	-0.0023	-0.0023	0.0023	0.0023
	5	0.2620	0.2620	0.2020	0.2020
	R	2	2	1	1
	N	0.1146	0.1146	-0.1146	-0.1146
	8	0.0200	0.0200	0.1980	0.1980
	R	2	2	1	1
	N	-0.5583	-0.5583	0.5583	0.5583
	Total R	6	6	6	6
	N	-1.0850	-1.0850	0.4850	0.4850
	Final Ranks	1	1	2	2

N: Normalized Values

R: Ranks

Table 4.11 Objective Function, Ranks, and Normalized Values of Storm of 7/30/1970

	i \ j	1	2	5	8
Upper Ross-Ade Watershed	1	99.35	118.0	153.5	118.0
	R	1	2	3	2
	N	-0.1651	-0.0304	0.2259	-0.0304
	2	63.72	59.75	68.12	59.75
	R	2	1	3	1
	N	0.0126	-0.0438	0.0751	-0.0438
	5	0.6217	0.0953	0.0036	0.0953
	R	3	2	1	2
	N	1.1522	-0.2997	0.5527	-0.2997
	8	0.0106	0.0098	0.0146	0.0098
	R	2	1	3	1
	N	-0.0471	-0.1099	0.2671	-0.1099
	Total R	8	6	10	6
	N	0.9526	-0.4874	0.0154	-0.4874
	Final Ranks	3	1	2	1

N: Normalized Values

R: Ranks

Table 4.12 Objective Function, Ranks, and Normalized Values of Storm of 9/04/1970

	i \ j	1	2	5	8
Upper Ross-Ade Watershed	1	93.60	95.55	175.9	95.52
	R	1	3	4	2
	N	-0.1596	-0.1452	0.4502	-0.1454
	2	298.1	270.5	299.0	296.9
	R	2	1	4	3
	N	0.0168	-0.0619	0.0257	0.0193
	5	0.1255	0.1583	0.0080	0.1539
	R	2	4	1	3
	N	0.0983	0.3275	-0.7226	0.2967
	8	0.0326	0.0085	0.0478	0.0084
	R	3	2	1	1
	N	0.2501	-0.4739	0.7007	-0.4769
	Total R	8	10	13	9
	N	0.2056	-0.3535	0.4540	-0.3036
	Final Ranks	3	1	4	2

N: Normalized Values

R: Ranks

Next, merit numbers were assigned for all the storm events and these are listed in Table 4.13. If the column totals were the same for some objective functions then the same merit number was assigned to these storms. The results in Table 4.13 shows that objective function 8 gives the best overall performance among the objective functions investigated. The performances of objective functions 1 and 2 are close to that of objective 8.

4.4.2. Results from the Second Set of Objective Functions

Another set of objective functions which includes some of those which have been used previously in hydrologic modeling was also investigated. The set consists of five objective functions which are listed below:

$$W_1=\Sigma(Y_i-X_i)^2 \qquad (4.21)$$

$$W_2=\Sigma((Y_i-X_i)/X_i)^2 \qquad (4.21)$$

$$W_3=\Sigma[\alpha((Y_i-X_i)^2)]+(1-\alpha)(Vobs-Vcal)^2 \qquad (4.23)$$

$$W_4=\Sigma[\alpha[\log_{10}Y_i-\log_{10}X_i]^2]+(1-\alpha)[\log_{10}Vobs-\mathrm{logt}Vcal]^2 \qquad (4.24)$$

$$W_5=\Sigma(\log_{10}Y_i-\log_{10}X_i)^2 \qquad (4.25)$$

Table 4.13 Merit Numbers Assigned to Storm Events, First Set of Objective Functions

	Oakdale Avenue Basin						Upper Ross-Ade Watershed			SUM
	5/19/59	7/02/60	7/26/60	10/14/60	4/19/63	4/29/63	7/18/70	7/30/70	9/04/70	
1	1	1	2	1	3	2	1	3	3	17
2	2	3	3	2	1	4	1	1	1	18
5	4	4	4	3	2	3	2	2	4	28
8	3	2	1	2	1	1	2	1	2	15

where Y_i: the observed hydrograph ordinates

X_i: the calculated hydrograph ordinates

The storm event of April 29, 1963, recorded on the Oakdale Avenue Basin was used to test the redundancy in objective functions. The optimal parameter values for this storm event are listed in Table 4.14 and the corresponding E_{ij} matrix is shown as Table 4.15. It is obvious that the rank matrix can be reduced to a 3×3 matrix since W_2 can be used to represent W_4 and W_5. The new rank matrix formed with W_1, W_2, and W_3 is shown in Table 4.16. The total column sum in Table 4.16 indicates that W_3 gives the best performance among these three objective functions for this storm.

Another five storms from the Oakdale Avenue Basin and the three storms from the Upper Ross-Ade Watershed were also used to investigate the objective functions. The respective elements E_{ij} computed from these storms and rank matrices are given in Tables 4.17 through 4.24. Merit numbers were assigned to the storm events based on the results from the rank matrices and these are shown in Table 4.25. The objective function W_1 which is the sum of the squared deviations, produces the smallest merit number sum and can be considered as the best objective function investigated. Objective function 3 in which the shapes, peaks and volumes are considered, is the second best objective function.

Table 4.14 The Optimal Parameter Values Obtained for Storm of 4/29/1963

Objective Function	Optimal Parameter Values					
	ABSTRT	DEPG	FI	FO	FC	KIF
1	0.067	0.050	0	7.933	0.279	2.407
2	0.200	0.197	0	7.949	0.399	1.832
3	0.000	0.050	0	7.933	0.279	2.407
4	0.200	0.197	0	7.949	0.399	1.832
5	0.200	0.197	0	7.949	0.399	1.832

Table 4.15 The E_{ij} Values of Storm 4/29/1963

i \ j	1	2	3	4	5
1	29.84	67.93	32.75	67.93	67.93
2	6458917	753.39	6462689	753.39	753.39
3	63353864	94220224	50118660	94220224	94220224
4	20.40	6.58	20.65	6.58	6.58
5	40.79	13.12	41.28	13.12	13.12

Table 4.16 Objective Function, Ranks, and Normalized Values of Storm of 4/29/1963

	i \ j	1	2	3
Oakdale Avenue Basin	1	29.84	67.93	32.75
	R	1	3	2
	N	-0.2506	0.4479	-0.1973
	2	6458917	753	6462689
	R	2	1	3
	N	0.3461	-0.6927	0.3467
	5	63353864	94220224	50118660
	R	2	3	1
	N	-0.0707	0.3008	-0.2300
	Total R	5	7	6
	N	0.0248	0.0560	-0.0806
	Final Ranks	2	3	1

N: Normalized Values

R: Ranks

Table 4.17 Objective Funct..n, Ranks, and Normalized Values of Storm of 5/19/1959

	i \ j	1	2	3
Oakdale Avenue Basin	1	12.96	24.19	51.41
	R	1	2	3
	N	-0.4202	-0.1353	0.5555
	2	63.86	16.25	86.30
	R	2	1	3
	N	0.1146	-0.5347	0.4200
	5	197710	10254802	25.97
	R	2	3	1
	N	-0.4621	0.9502	-0.4899
	Total R	5	6	7
	N	-0.7677	0.2820	0.4856
	Final Ranks	1	2	3

N: Normalized Values

R: Ranks

Table 4.18 Objective Function, Ranks, and Normalized Values of Storm of 7/02/1960

	j i	1	2	3
Oakdale Avenue Basin	1	5.310	5.630	7.490
	R	1	2	3
	N	-0.1159	-0.0714	0.1873
	2	7.11	6.90	8.19
	R	2	1	3
	N	-0.0338	-0.0583	0.0922
	5	1007130	1441720	295438
	R	2	3	1
	N	0.0787	0.4375	-0.5142
	Total R	5	6	7
	N	-0.0730	0.3078	-0.2347
	Final Ranks	2	3	1

N: Normalized Values

R: Ranks

Table 4.19 Objective Function, Ranks, and Normalized Values of Storm of 10/14/1980

	j i	1	2	3
Oakdale Avenue Basin	1	18.29	38.68	32.58
	R	1	3	2
	N	-0.3209	0.2451	0.0758
	2	20130	19297	45320
	R	2	1	3
	N	-0.2267	-0.2499	0.4766
	5	4291886	11458105	574478
	R	2	3	1
	N	-0.1373	0.7187	-0.5814
	Total R	5	7	6
	N	-0.6849	0.7139	-0.0290
	Final Ranks	1	3	2

N: Normalized Values

R: Ranks

Table 4.20 Objective Function, Ranks, and Normalized Values of Storm of 7/28/1960

	j / i	1	2	3
Oakdale Avenue Basin	1	20.07	20.24	25.36
	R	1	2	3
	N	-0.0715	-0.0648	0.1363
	2	93.40	91.50	146.38
	R	2	1	3
	N	-0.1297	-0.1442	0.2739
	5	16860093	17730403	11358889
	R	2	3	1
	N	0.2978	0.3502	-0.0331
	Total R	5	6	7
	N	0.0966	0.1412	0.3771
	Final Ranks	1	2	3

N: Normalized Values

R: Ranks

Table 4.21 Objective Function, Ranks, and Normal...ed Values of Storm of 4/19/1963

	j / i	1	2	3
Oakdale Avenue Basin	1	92.60	113.8	100.8
	R	1	3	2*
	N	-0.0825	0.0960	-0.0136
	2	18.89	16.81	48.62
	R	2	1	3
	N	-0.2467	-0.3091	0.5566
	5	4673392	35803040	245
	R	2	3	1
	N	-0.3537	0.8949	-0.5412
	Total R	5	7	6
	N	-0.6837	0.6818	0.0018
	Final Ranks	1	3	2

N: Normalized Values

R: Ranks

Table 4.22 Objective Function, Ranks, and Normalized Values of Storm of 7/18/1970

	i \ j	1	2	3
Upper Ross-Ade Watershed	1	492.6	9772	537.7
	R	1	3	2
	N	-0.4590	0.9114	0.2623
	2	218.2	60.30	77.73
	R	3	1	2
	N	0.6133	-0.3604	-0.2529
	5	42964540	3219207310	25497663
	R	2	3	1
	N	-0.4717	0.9512	-0.4795
	Total R	6	7	5
	N	-0.3174	1.5022	-0.4701
	Final Ranks	2	3	1

N: Normalized Values

R: Ranks

Table 4.23 Objective Function, Ranks, and Normalized Values of Storm of 7/30/1970

	i \ j	1	2	3
Upper Ross-Ade Watershed	1	99.00	128.0	153.0
	R	1	2	3
	N	-0.1860	0.0089	0.1770
	2	3688	59.00	3527
	R	3	1	2
	N	0.3640	-0.6817	0.3178
	5	3581183	30031602	12076
	R	2	3	1
	N	-0.3651	0.9010	-0.5359
	Total R	6	6	6
	N	-0.1871	0.2282	-0.0413
	Final Ranks	1	3	2

N: Normalized Values

R: Ranks

Table 4.24 Objective Function, Ranks, and Normalized Values of Storm of 9/04/1970

	i \ j	1	2	3
Upper Ross-Ade Watershed	1	76.00	98.0	173.0
	R	1	2	3
	N	-0.2776	-0.1236	0.4012
	2	240752	55113	97692819
	R	2	1	3
	N	-0.4783	-0.4811	0.9594
	5	6625618	9910826	38668
	R	2	3	1
	N	0.1355	0.5399	-0.6754
	Total R	5	6	7
	N	-0.6204	-0.0648	0.6852
	Final Ranks	1	2	3

N: Normalized Values

R: Ranks

Table 4.25 Merit Numbers Assigned to Storm Events, Second Set of Objective Functions

	Oakdale Avenue Basin						Upper Ross-Ade Watershed			SUM
	5/19/59	7/02/60	7/26/60	10/14/60	4/19/63	4/29/63	7/18/70	7/30/70	9/04/70	
1	1	2	1	1	1	2	2	1	1	12
2	2	3	2	3	3	3	3	3	2	24
3	3	1	3	2	2	1	1	2	3	18

4.5 Summary

Two sets of objective functions which may be used in urban rainfall-runoff models were investigated by using the procedure developed by Diskin and Simon (1977). ILLUDAS was the rainfall-runoff model used in the present study. The following general conclusions emerge from this study.

(1). The subjectivity involved in the selection of the objective functions may be reduced by using the analysis such as that discussed above.

(2). The sum of the squared deviations between the observed and calculated hydrograph ordinates has been the most frequently used objective function in the past and the results of present study show that this objective function gives best overall performance.

(3). Although the results presented are data specific, the trends are clear. The fact that several objective functions give identical parameter estimates is also of significance.

(4). The requirement that the diagonal elements E_{ii} should be the minimum value among the row elements can be used to check whether the global optimum is reached.

CHAPTER V

SUMMARY AND CONCLUSIONS

5.1 Introduction

Urban runoff models are required to handle the increasingly complex problems of urban stormwater management. Models of this kind have been developed in the past fifteen years or so. The recognition of the importance of accurate prediction of urban runoff quantity and quality is the main reason for developing these models. Many usable urban runoff models have been developed. However, the accuracy and the effectiveness of these models are still questioned. Several aspects of these models need further investigation. Two of these aspects are dealt with in this study. They are, (1) optimal parameter estimation in urban runoff models, and (2) analysis of the objective functions used in optimal parameter estimation of rainfall-runoff models.

5.2 Optimal Parameter Estimation

Urban runoff models are developed either to design a new drainage system or to evaluate existing drainage systems. Only an appropriately calibrated model can be used to accurately evaluate a system. Effective and economical methods of calibration have not yet been developed although the importance of calibration of these models has long been recognized. At present, trial and error methods are used to calibrate most of the urban runoff models. In this study, Rosenbrock's optimal parameter estimation method is incorporated into two important urban runoff models, the ILLUDAS and Runoff Block of SWMM model. The parameter estimation method in these two models was tested by using data from the Upper Ross-Ade Watershed and the Oakdale Avenue Basin. Both the regeneration and prediction performances of calibrated models were investigated. The advantages of using optimal parameter estimation methods are brought out in this portion of the study. The third model compared is MINNOUR. The comparison of these three models reveals that the complexity of the model structure does not guarantee better performance. The simpler models, ILLUDAS and MINNOUR gave results which were superior to those given by SWMM in most cases.

5.3 Objective Function Analysis

An objective function is needed in parameter estimation methods to serve as an index of agreement between observed and calculated hydrographs. The selection of the objective function is important as it affects the parameter estimates and hence the outputs from the models. The methodology developed by Diskin and Simon (1977) is used in the study. Two sets of objective functions are investigated. First, the twelve general objective functions used by Diskin and Simon (1977)) and second, five objective functions which have been used in rainfall-runoff models. They are tested by using ILLUDAS and data from the Upper Ross-Ade Watershed and the Oakdale Avenue Basin. The study shows that the sum of the squared deviations between the observed and computed flows gave the best overall performance and is recommended for further use.

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